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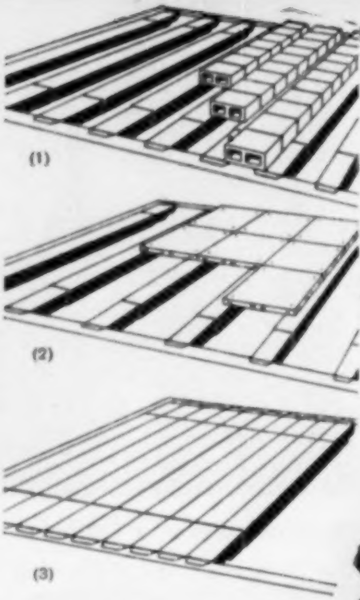

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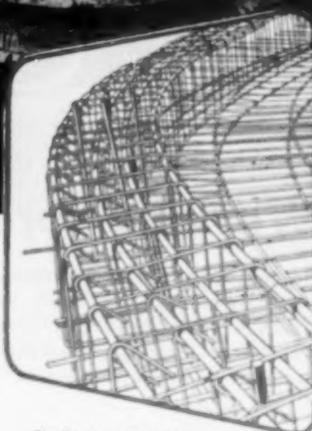

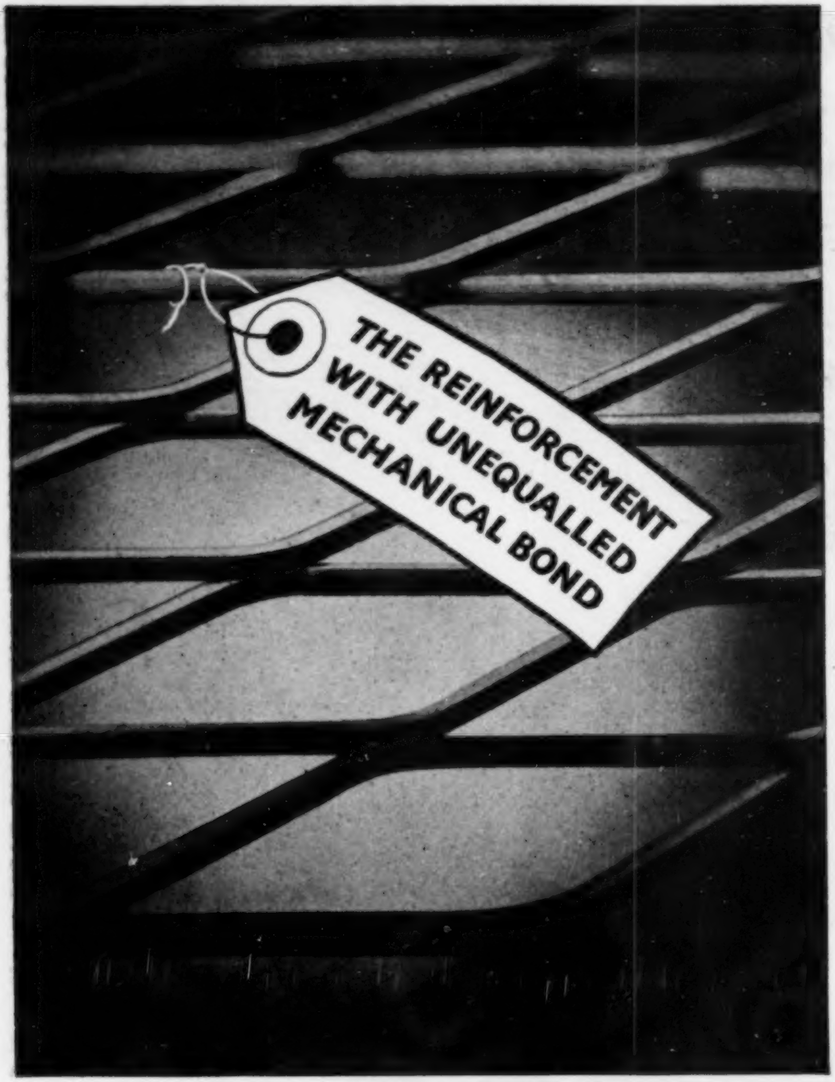
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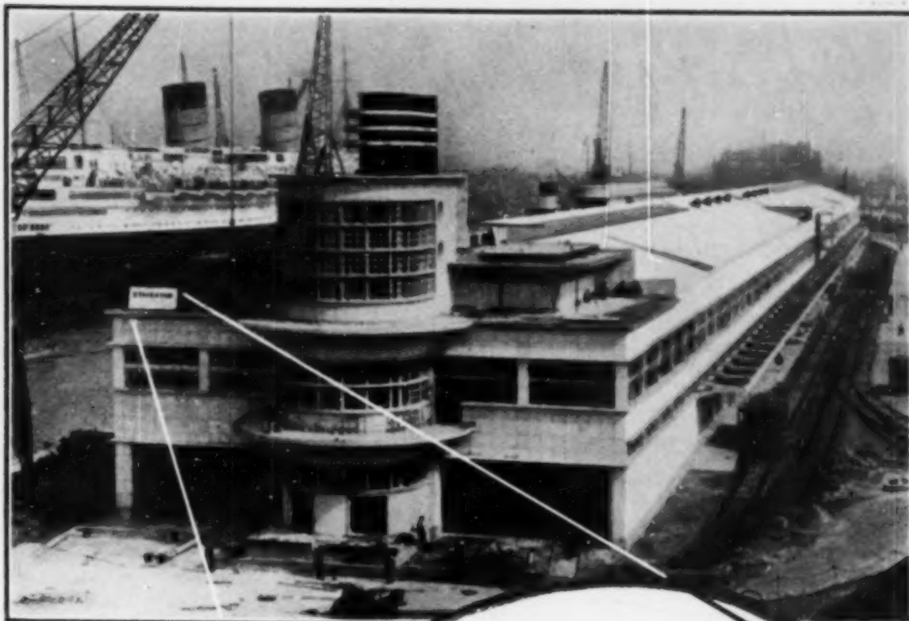

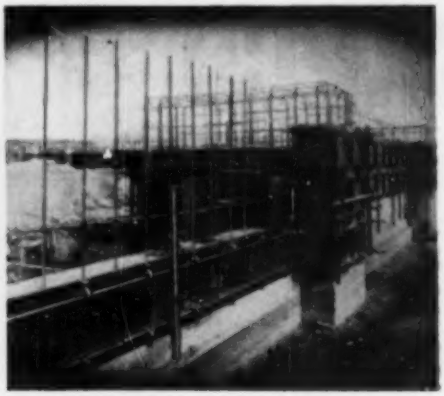

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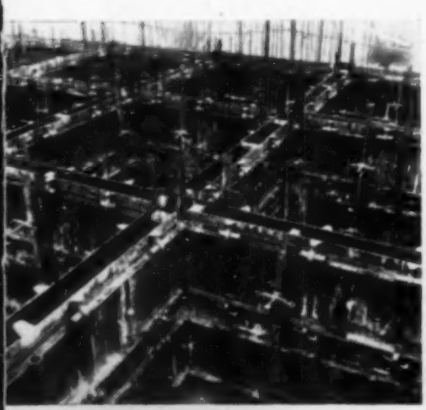

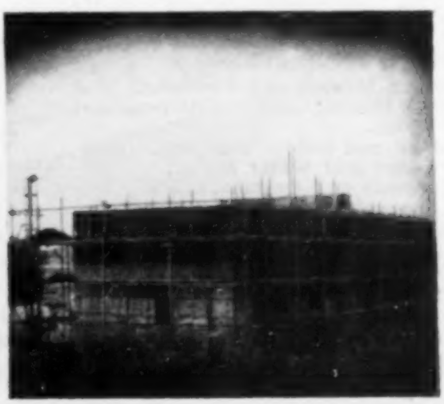

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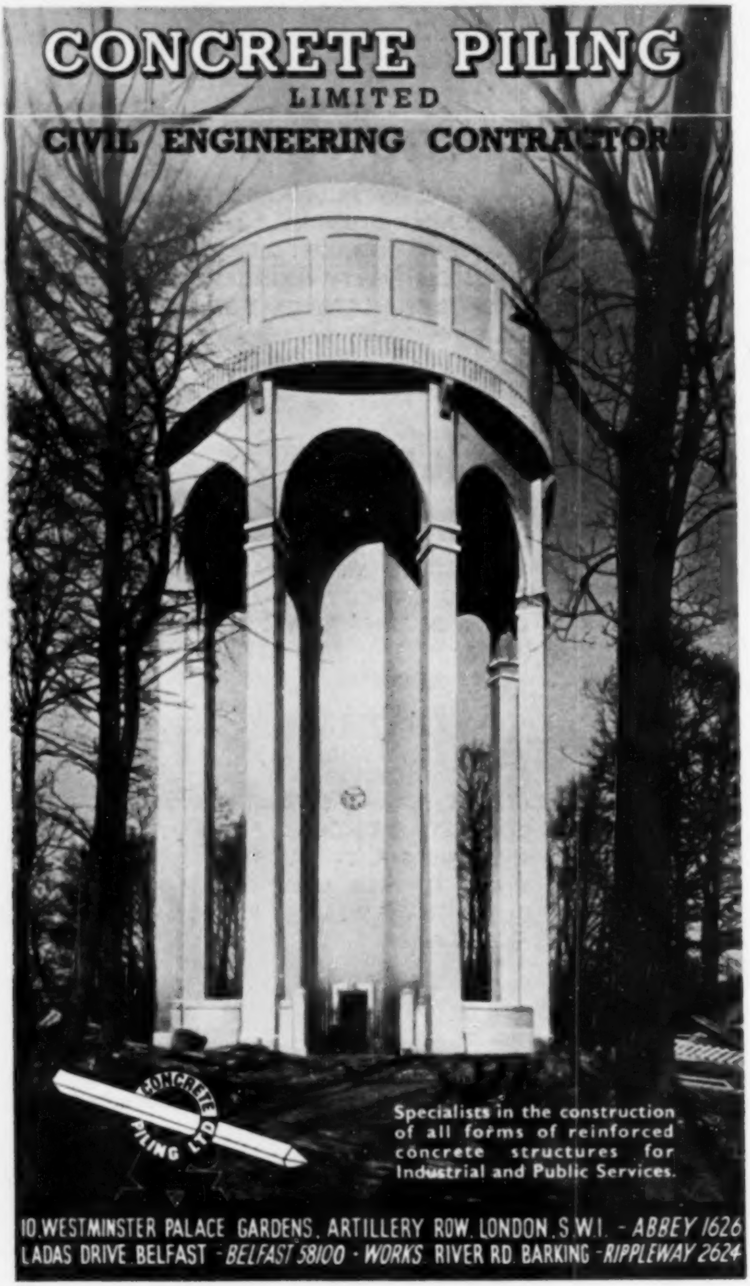
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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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EDITORIAL NOTES

The Confirmation of Structural Theories by Tests.

TRENDS in the development of the design of reinforced concrete structures wherein theory is confirmed by tests are evident in the contributions relating to concrete in the final report of the third congress of the International Association for Bridge and Structural Engineering which was published recently. (Abstracts from the preliminary report were given in this journal for October, 1948.) Few of the contributors deal with concrete and reinforcement as structural materials, but several are concerned with prestressed concrete, others with structural safety, and a few with structures of ordinary design. Many contributors, whose papers are probably those of greatest and widest interest, attempt to solve problems that have proved to be intractable, such as those concerned with arch bridges and dams, flat slabs, slabs spanning in two directions, and thin-slab structures including "shells", prismatic roofs, and walls acting as beams. Some of the solutions are entirely mathematical, others are based on experiments on models, and others on the behaviour of actual structures.

An ideal in structural analysis is that the results of these three methods of approach should be in agreement. Such accordance is a distinguishing feature of the investigation of arch bridges in which the deck constructed monolithically with the arch affects the action of the arch. An example [(1)*] is an open-spandrel arch-rib bridge of about 450 ft. span and 150 ft. rise for which the calculated deformations compare very favourably with those measured on three-dimensional celluloid models and on the structure under test loads. Similar agreements which were obtained for a bridge of different construction,[(2)] a solid-spandrel arch-slab of 50 ft. span and a rise of less than 8 ft., show the possibility of accurately analysing complex structures by simple laboratory methods. More elaborate apparatus is required for testing models of arch dams, and measurements on actual structures may not be easy. Tests,[(3)] however, show concordance between the behaviour of model dams and the more complete theories. One such theory [(4)] is that of the late Professor Ritter, which is briefly described in the report and which takes into account the deformation of the rock supporting the dam.

The results of tests on the whole of a structure, or on full-sized replicas of structural parts are, of course, more convincing than tests on models if the restraints to which the part would be subjected in the structure act on the part

* The references relate to the bibliography on page 234 of this number.

under test. Of the latter type are the tests [5] on 4-in. load-bearing single-story walls, which failed by crushing and not by buckling. One deduction from these tests is that the ratio of the compressive stress in an axially-loaded wall at failure to the crushing strength of 4-in. cubes is between two-thirds and three-quarters, the smaller ratio applying to walls that are short compared with their height. Another contributor [6] shows that a wall acting as a beam, that is a thin structural element that is deep in relation to its span and to which the common theory of beams is inapplicable, can be analysed by consideration of virtual work, the results being confirmed by the deformations of deep beams.

Other thin-slab constructions considered include a method of designing prismatic structures [7] on the assumption that all tensile forces are resisted by the reinforcement. This method is unrelated to tests, as are also two of the purely mathematical analyses [8] of "shell" roofs, but observations [9] made during the demolition of a large double circular "shell" roof (damaged by explosions) should be of great value to designers of structures of this type, the design of which depends almost entirely on theoretical analyses embodying mathematics of a high degree and on assumptions that may or may not be true. As one contributor [10] points out, the forces and bending moments in a "shell" are calculated according to the theory of elasticity but the tensions are aggregated to give a tensile force which, when divided by the permissible tensile stress, gives the area of reinforcement, ignoring the fact that the strain assumed in the elastic analysis and that due to the working stress are not alike as the working stress is not uniform over the tensile zone. This serious discrepancy is stated to nullify the basis of the precise mathematics, and it is then postulated that a simple method of calculation can be derived from a study of the conditions at failure. A somewhat similar basis is that of the analysis [11] of slabs spanning in two directions by consideration of the pattern of the cracks at failure combined with plastic conditions at this stage.

Although tests on actual structures are, where practicable, desirable to confirm theories, it is essential that the conclusions deduced from the results of the tests should be reliable. In this connection the tests made many years ago to establish the empirical coefficients for the bending moment on flat slabs are an example. As explained [12] the bending moments to which the slabs were subjected were computed from measurements of the elongation of the reinforcement. The actual moment of resistance of a reinforced concrete slab is a combination of the moments of resistance contributed by the tensile stresses in the reinforcement and that in any uncracked concrete below the neutral axis. If the bending moment is deduced from the former moment of resistance only it is too small, and explains partly the divergence between such results and the results obtained from carefully-derived theories. The earlier theories of flat slabs disregarded the change of section due to the column-head, or included this effect very approximately, and defective analyses, some of which agree nevertheless with the incorrect measurements of actual structures, were the consequence. What appears to be an improved theory has been derived, but tests on models and actual structures are necessary, and are proceeding, to confirm the accuracy of the theoretical results.

It is a healthy sign that less dependence seems to be put on the results of pure mathematics unless they are confirmed by tests, not because the mathematical procedure is itself faulty but because the premises do not correspond to actualities.

New Passenger Terminal, Ocean Dock, Southampton.

THE new Ocean Terminal building (*Figs.* 1 and 2), Southampton Docks, is 1272 ft. 6 in. long and has a normal width of about 94 ft. extended by galleries and the like to about 111 ft. Although the main structure, the construction of which is approaching completion, is of unencased steelwork and incorporates a pitched two-pin portal frame roof of 92 ft. span, there is a considerable amount of cast-in-situ and precast plain and reinforced concrete in the foundations, balconies, galleries, and floors. Cross sections of the building are shown in *Fig.* 4.

The site is made-up ground overlying ballast. The structure is therefore supported on 625 concrete piles, formed in situ with precast cylindrical sections, penetrating generally to a depth of 35 ft. to the ballast. As the level of the ballast varies some of the piles are as short as 18 ft., whereas others are up to 65 ft.

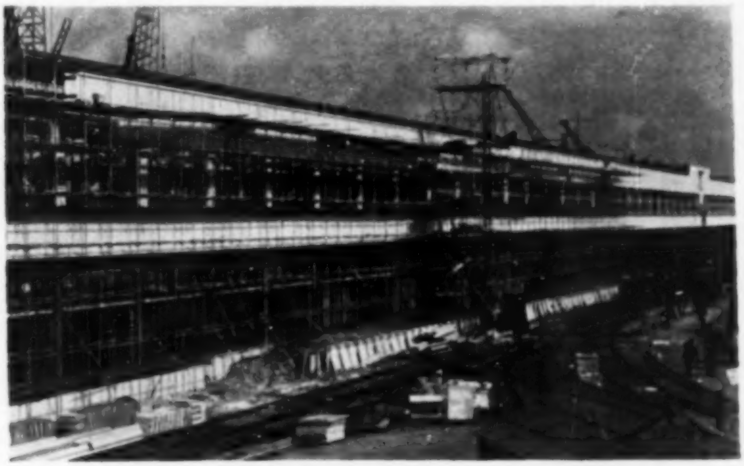

Fig. 1.—Elevation showing Precast-Slab Walls.

Fig. 2.—East Elevation showing Canopy.

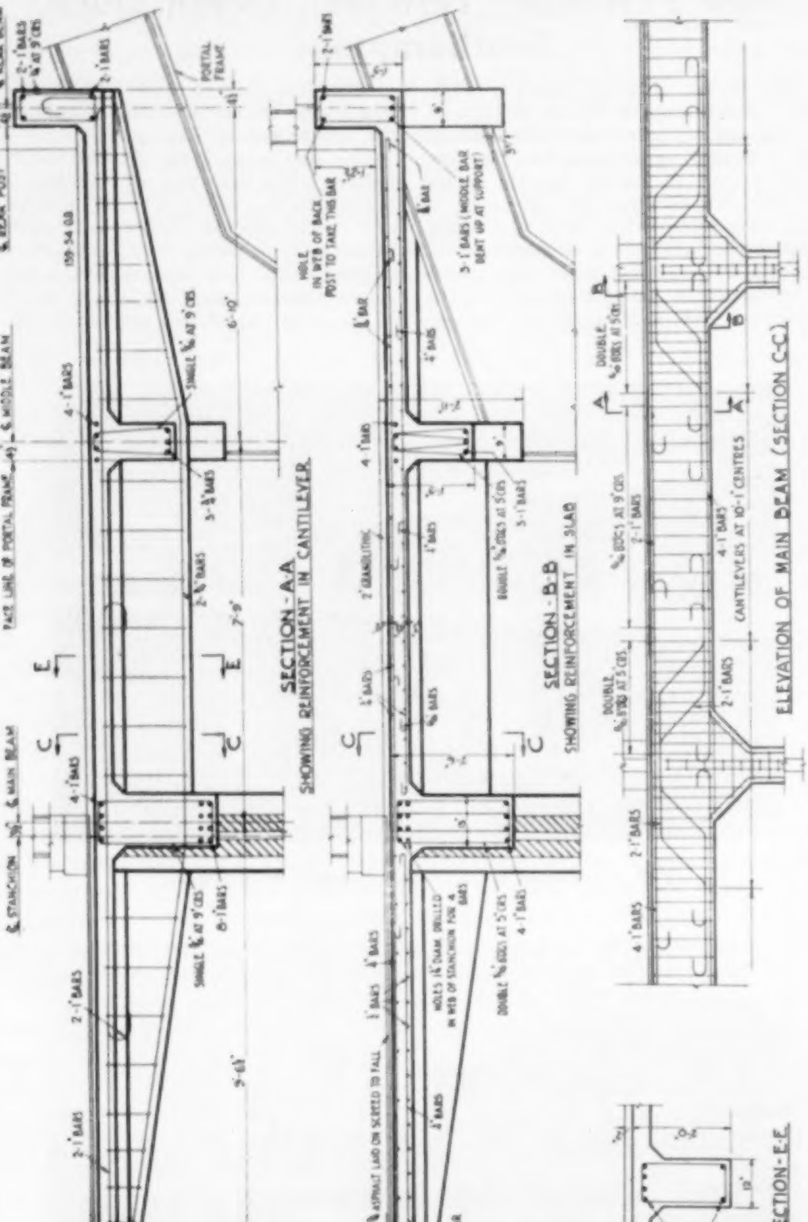

Fig. 3.—Details of Reinforcement in Balcony.

long where pockets of sand and clay occur. Two sizes of pile are used, 17½ in. diameter and 20 in. diameter, the greatest working loads being 60 tons and 90 tons respectively. Test loads of 120 tons and 180 tons were applied to two of the piles bearing on the ballast, and four piles bearing on sand and clay were subjected to test loads representing an overload of 50 per cent. Pile-driving commenced in the spring of 1947 and the foundations were nearly completed about the end of that year. Erection of the steelwork commenced in February, 1948, and was completed in November, 1948.

The ground floor, which is laid on hardcore rolled into the made-up ground, is a concrete slab having a nominal thickness of 5 in. and laid generally to a slope of 1 in 20. Waterproofed paper is laid on the hardcore and the slab is reinforced by a 6-in. square mesh of cold-drawn wire placed about 1 in. from the bottom. The concrete is mixed in the volumetric proportions of 1 : 1½ : 3 but is batched on the basis of 1-cwt. of cement. It is carried from the mixer in a motor-skip containing a complete batch of concrete. One part of the ground floor is occupied by a railway station comprising an island platform for passengers and, on one side of the gullet line (*Fig.* 4), a platform for unloading goods. The passenger platforms are constructed with standard precast concrete members made at the concrete depot of British Railways, Southern Region, at Exmouth Junction. As these members, particularly the platform coping slabs, are not designed to withstand the blows and loads entailed when unloading cargo from wagons, the goods platform is of cast-in-situ reinforced concrete with a protective covering of 3 in. of creosoted timber. The rails in the gullet are laid on ordinary reinforced concrete sleepers of the Ministry of Transport type. The gullet was excavated by a mechanical shovel discharging into motor lorries.

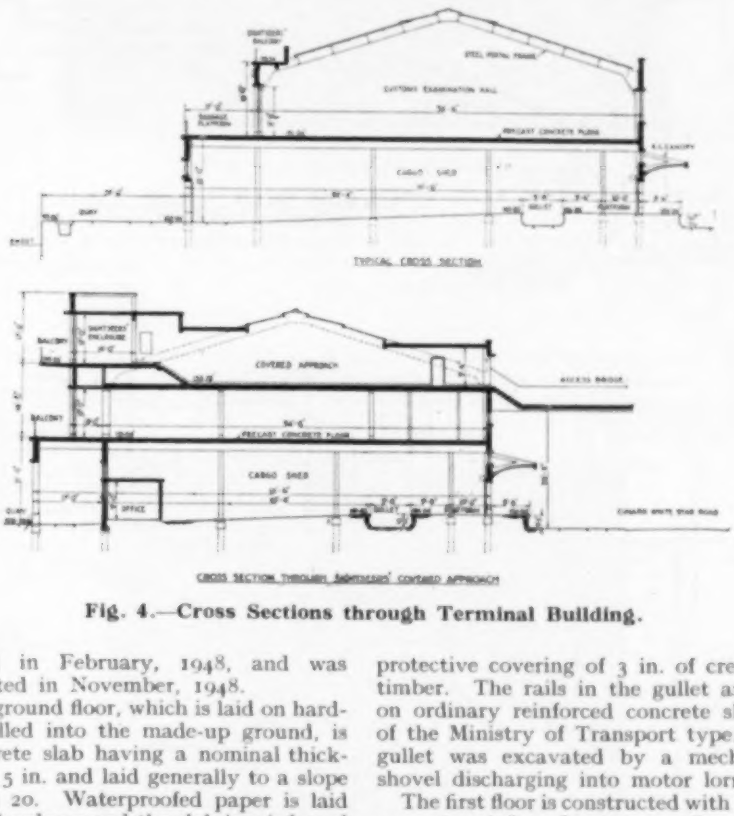

Fig. 4.—Cross Sections through Terminal Building.

The first floor is constructed with hollow precast reinforced concrete floor slabs supported on steel beams. The parts of this floor liable to be loaded with baggage, such as baggage platforms and customs examination rooms, are designed for a superimposed load of 200 lb. per square foot, but elsewhere the floor is designed for 100 lb. per square foot. Flat roofs in the open are designed for a superimposed

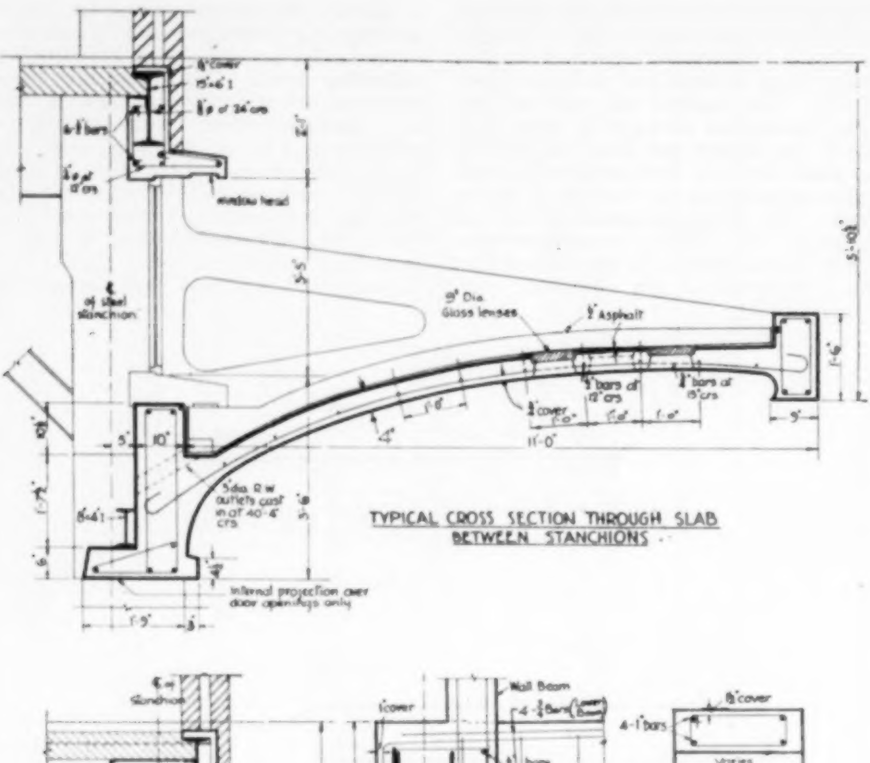

Fig. 5.—Details of Canopy.

Fig. 6.—Underside of Canopy.

load of 30 lb. per square foot, but similar roofs covering structures within the main building are designed for 50 lb. per square foot.

Balconies.

Along part of the western side of the building there is a reinforced concrete balcony for sightseers. The balcony is cantilevered from the main building (*Fig.* 4), and details of the reinforcement in the slab and in a typical transverse cantilevered beam supported on three longitudinal reinforced concrete beams are shown in *Fig.* 3. The reinforcement in the main longitudinal beam, which is supported on the outer row of steel stanchions encased in reinforced concrete,

Fig. 7.—Top of Canopy.

is also shown. The stanchions are at 20-ft. 2-in. centres, but the transverse beams are at 10-ft. 1-in. centres and occur at the two outer quarter-points of the span of the main longitudinal beam. The middle and rear longitudinal beams are supported at 20-ft. 2-in. centres on the steel portal frames. The concrete is high-grade 1 : 1½ : 3 concrete.

Steel shutter plates supported on telescopic steel joists are mainly used for the construction of the balcony, although timber is used for those parts where the dimensions and shapes do not permit the use of standard plates. Timber shuttering is for this reason also used for the reinforced concrete and concrete encasing the steelwork of the tower at the end of the building seen in *Fig.* 2.

The four staircases giving access to the first floor and balcony are constructed in cast-in-situ concrete reinforced generally with twisted square steel bars. The curtain walls around the stairwells are reinforced with two layers of small square mesh, and steel shuttering is used in their construction.

Prestressed Concrete Footbridge.

Independent access to the sightseers' balcony will be provided by a prestressed concrete footbridge (*Fig.* 9) having a span of 69 ft. and an overall width of 11 ft. 6 in. The bridge will have a slope of 1 in 9. The two main prestressed concrete girders, which will have a depth of 6 ft. 6 in., will comprise precast concrete panels, and will each contain five cables secured at their ends in anchorage cones of the Freyssinet type. The deck of the bridge will be constructed of hollow precast concrete floor beams similar to those in the main building and will be finished with 2 in. of granolithic. Windows will be provided above some of the panels. Each precast panel will be 6 ft. 2 in. long and 6 ft. 6 in. high, and will comprise a web 4 in. thick, a top flange 10 in. wide, and a bottom flange 10 in. wide. The panels are lightly reinforced for handling purposes. It is intended to assemble and prestress the girders on the ground before hoisting them into position.

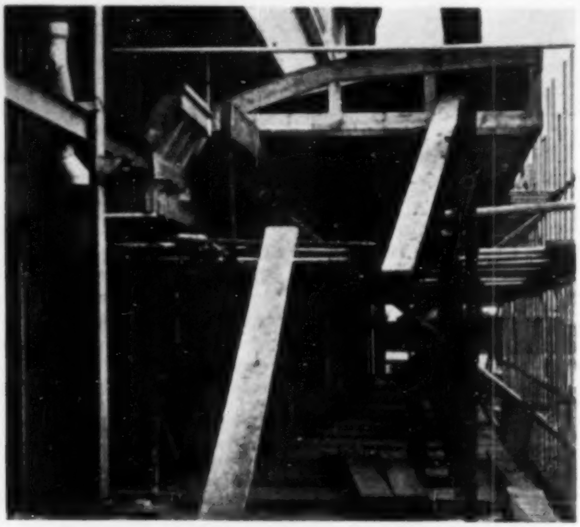

Fig. 8.—Shuttering for Canopy.

Canopy.

Throughout the length of the building there is being constructed on the eastern side a reinforced concrete canopy (*Figs.* 6 and 7), curved in cross section, projecting horizontally 11 ft., and supported by cantilevers projecting above the curved slab. Details of the reinforcement in the

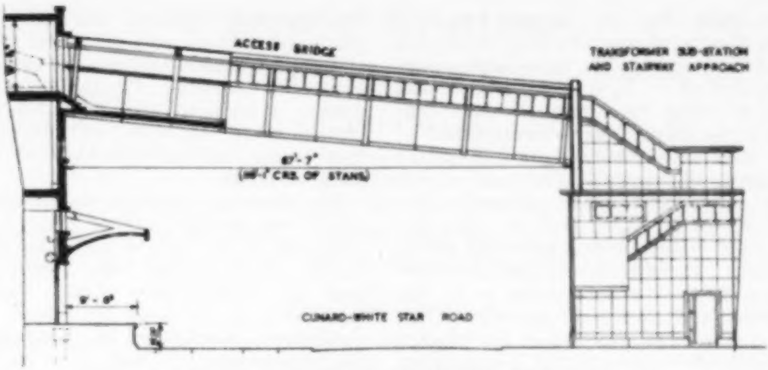

Fig. 9.—Prestressed Concrete Bridge.

slab and cantilevers are shown in *Fig.* 5. The cantilevers are provided at each steel stanchion of the main building, and the tensile reinforcement is anchored behind the inner flange of the stanchions. The slab is 4 in. thick and is reinforced mainly to span transversely from a longitudinal wall beam to a beam along the outer edge of the canopy. The wall beam is supported on the steel stanchions and the edge beam on the ends of the cantilevers. Circular glass lenses are provided in the slab and the main transverse reinforcement comprises one $\frac{3}{4}$-in. mild steel bar passing between the lenses, the positions of which are staggered so that the bars are at 13-in. centres.

The shuttering for the canopy (*Fig.* 8) is made up of parts each 5 ft. long and extending almost the entire width of the canopy. The root of the canopy, which forms one side of the longitudinal wall beam, is shuttered with a separate part that extends the entire distance between the stanchions, which are at 20-ft. 2-in. centres. Each of the curved parts of shuttering for the slab comprises four wooden templates between which span 2-in. by $\frac{3}{4}$-in. slats to which are attached sheets of pressed hard-board shaped to the curvature of the soffit of the slab.

Fig. 10.—Construction of Wall.

Walls.

Where they are not occupied by sliding doors, the sides of the building (*Figs.* 1 and 2) are mainly glazed, but where they are filled the walls are constructed of precast concrete slabs surmounting, at ground level, a cast-in-situ reinforced concrete plinth. The window sills are also precast, but lintels are of cast-in-situ reinforced concrete in combination with steel beams as shown in the typical cross section through the slab of the canopy in *Fig.* 5.

The walls (*Fig.* 10) comprise two 4-in. leaves of rebated concrete slabs separated by a 2-in. cavity. The inner leaf is constructed of 18-in. by 9-in. by 4-in. blocks of light-weight insulating sawdust concrete. The outer leaf is constructed of dense concrete slabs having a ¾-in. facing composed of crushed Portland stone, white sand, and white cement. The width of the standard slabs is 2 ft., but the height varies, being generally about 2 ft. 5 in. There are about 4500 standard slabs and many others made to fit the steelwork and structural concrete. The total quantity of concrete in the precast slabs exceeds 22,000 cu. ft. The backing concrete of the slabs is about 3¼ in. thick and is generally composed of 3½ parts (by volume) of ¾-in. graded crushed ballast, 1 part of grit, and 1 part of rapid hardening Portland cement. A very dry mixture is used, the amount of water added being varied to suit the moisture content of the ballast which sometimes comes straight from the washing plant. Lifting eyes (*Fig.* 10) are cast in the slabs for hoisting and bedding the slabs. The joints are rebated and are set in mortar.

Two expansion joints are provided across the building. The joints, at which double stanchions are provided, completely sever the walls and floors, and those in the walls are sealed by a V-shaped strip of copper.

The structural work at the Ocean Terminal was designed and is being erected under the supervision of Mr. J. H. Jellett, O.B.E., M.A., M.Inst.C.E., Docks Engineer at Southampton of the Southern Region of the Railway Executive. The prestressed footbridge is designed by the Pre-stressed Concrete Co., Ltd. The general contractors are Staverton Builders, Ltd., and the piles were driven by West's Piling and Construction Co., Ltd. The hollow floor slabs are supplied and laid by Concrete, Ltd. The slabs for the outer leaf of the hollow walls are made by Blokcrete, Ltd., and those for the inner leaf by Lignacite (Fordingbridge), Ltd. The steelwork was fabricated and erected by the Cargo Fleet Iron Co., Ltd.

The Final Report of the Third Congress of the Association for Bridge and Structural Engineering.

THE following are the papers in the final report of the third congress of the Association for Bridge and Structural Engineering referred to in our Editorial Note. The papers are written in the languages indicated, but a summary of each in English is given.

1. "The Fürstenland Bridge at St. Gall (Switzerland)." By Professor K. Hofacker (Zurich). p. 421. In German.
2. "Uniting the Floor (Deck) with the Arch of a Flat (low-rise) Bridge." By Dr. K. Waitzmann (Prague). p. 453. In French.
3. "Elastostatic Tests on Models of Arched Dams." By Professors H. Beer and E. Tschech (Graz). p. 627. In German.
4. "Contribution to the Design of Arched Dams." By Dr. P. Lardy (Zurich). p. 623. In French.
5. "The Strength of Thin Concrete Walls in Axial Compression under Distributed Loading." By A. E. Seddon. p. 589. In English.
6. "The Application of the Virtual-Work Equation for Calculating Wall Beams." By Professor J. Mandes. p. 607. In English.
7. "The Limit Design of Shells (Prismatic Thin-slab Structures)." By Dr. G. De Kazinczy (Stockholm). p. 615. In German.
8. "Introduction of a General Theory of Shells of Translation." By L. Broglio (Rome). p. 553. In French.
   "On Integration of the Differential Equation for Thin Shells without Bending." By K. W. Johansen (Copenhagen). p. 597. In English.
9. "Report on Thin Slabs (shells) Constructed in Spain." By Professor E. Torroja (Madrid). p. 575. In French.
10. "Critical Notes on the Calculation and Design of Cylindrical Shells." By K. W. Johansen (Copenhagen). p. 601. In English.
11. "The Ultimate Strength of Reinforced Concrete Slabs." By K. W. Johansen (Copenhagen). p. 565. In English.
12. "The Calculation of Flat Slab Floors." By Dr. A. M. Haas ('s-Gravenhage). p. 535. In English.

Prismatic Structures.

We have received the following communication from Mr. Minglung Pei, Ph.D., of Easton, Penna., U.S.A.

I have seen the articles on prismatic thin-slab structures by Mr. A. J. Ashdown published in your journal for October 1948 to August 1949, and I am happy to see that this type of construction has been brought to the attention of engineers. It appears, however, that Mr. Ashdown has unwittingly made some mis-statements. The following deals with the first article on prismatic thin-slab structures of one span and the article on multiple-span structures.

Shearing Stresses.

Referring to *Fig.* 1 the following formulæ are given for the normal and shearing stresses in prismatic thin-slab structures of one span:

$$f_{AB} = \frac{-2T_0 - 4T_1}{a_A} + \frac{M_A}{Z_A} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$f_{BA} = \frac{4T_1 + 2T_2}{a_B} + \frac{M_B}{Z_B} \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$\frac{T_0}{a_A} + 2T_1\left(\frac{1}{a_A} + \frac{1}{a_B}\right) + \frac{T_2}{a_B} = \frac{1}{2}\left(\frac{M_A}{Z_A} + \frac{M_B}{Z_B}\right) \quad . \quad . \quad . \quad (3)$$

$$S = \int P_B dx - \frac{d_B}{2}\left(\frac{dT_1}{dx} + \frac{dT_2}{dx}\right) \quad . \quad . \quad . \quad . \quad (4)$$

$$q = \frac{S}{a_B} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

where f_{AB} and f_{BA} are the normal stresses at edge 1 of slabs A and B respectively.

T_1 is the shearing force acting on the transverse section at edge 1, and is at right angles to the section, and induces compression in slab A and tension in slab B.

M_A is the moment due to the unbalanced load in the plane of slab A. (Viewed from fixed point at the side, clockwise moment is positive.)

a_A is the cross-sectional area of slab A.
Z_A is the section modulus of slab A.
d_B is the width of slab B.
S is the total shearing force along a transverse section of the slab.
P_B is the intensity of the unbalanced load.
q is the shearing stress.

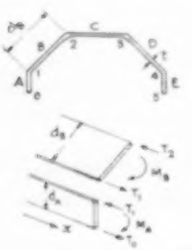

Fig. 1.

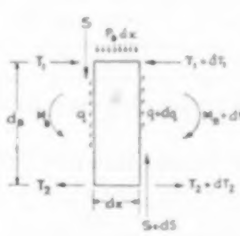

Fig. 2.

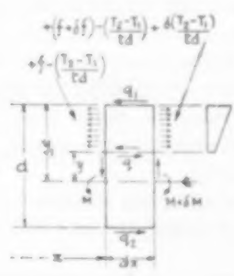

Fig. 3.

Equation (3) gives the values of T. Equations (1) and (2) give the values of the normal stresses. Equation (5) gives the value of shearing stress. Equations (1), (2), and (3) apply to multiple-span structures as well as structures of one span provided that only two slabs meet at a joint. For example, they apply to multiple-span roofs without intermediate ties, which are not essential.

Equations (4) and (5) for the shearing stresses are incorrect. Consider a strip of slab B (*Fig.* 2) between two transverse sections dx apart. By statics it is evident that $P_B\,dx = dS$ and $\int P_B\,dx = S$. The summation of the shearing stress over the section is $t\int_0^{d_B} q.dy = S$. The shearing stresses at the edges must be

$$q_1 = \frac{1}{t}.\frac{dT_1}{dx} \text{ and } q_2 = \frac{1}{t}.\frac{dT_2}{dx}.$$

Thus it is incorrect to assume that the shearing stress q is uniformly distributed along the width d_B. If the material is homogeneous and the plane section remains plane after bending, it has been shown [1] that the distribution over the width is parabolic, and the maximum shearing stress is

$$q_M = \frac{3S}{2a} - \frac{q_1 + q_2}{4} \quad . \quad . \quad . \quad . \quad . \quad (6)$$

For a reinforced concrete slab with reinforcement along its edges, the shearing stress may be distributed in other ways.[2] Due to the presence of T, the distribution is more favourable than in ordinary beams. In general, the shearing stress is not critical in thin-slab structures. Equation (4) was probably derived by Mr. Ashdown by taking moments about the middle of the strip (*Fig.* 2),

$$S.dx - dM_B + dT_1.\frac{d_B}{2} + dT_2.\frac{d_B}{2} = 0.$$

Therefore $S = \dfrac{dM}{dx} - \dfrac{d_B}{2}\left(\dfrac{dT_1}{dx} + \dfrac{dT_2}{dx}\right)$.

Assuming erroneously that $\dfrac{dM}{dx} = \int P_B.dx$, equation (4) is obtained.

MR. ASHDOWN replies:

In the footnote to my first article (October, 1948) attention is drawn to the fact that the formulæ for slabs of uniform thickness assume that the shearing stresses are uniformly distributed and that a partial parabolic distribution is given in the analysis of Professor G. Winter and Mr. M. Pei. It is agreed that the latter distribution is more correct and can be derived as follows.

It must be noted that the coplanar shearing force $\int P.dx$ is not altered by the imposition of the forces T, but the distribution of the shearing stresses may be affected.

The shearing stresses can be divided into two parts. The first part consists of the shearing stresses induced by the load P, and the distribution throughout

[1] G. Ehlers: "Die Spannungsermittlung in Flächentragwerken." Beton u. Eisen, Vol. 29, p. 281, 1930.
[2] G. Winter and M. Pei: "Hipped Plate Construction." Journal of American Concrete Institute, Vol. 18, No. 5, p. 516, January 1947.

the depth is the ordinary parabolic distribution, and the maximum shearing stress at the axis is

$$\frac{3\int P.dx}{2a} = \frac{3S}{2a}.$$

The second part is that induced by the shearing forces T_1 and T_2 imposed at the top and bottom of the slab respectively, and the change of axial thrust $(T_2 - T_1)$ along the x-axis. The shearing stress along the top edge (*Fig.* 3) is

$$q_1 = \frac{\delta T_1}{\delta x}.\frac{1}{t},$$

where t is the thickness of the slab. Along the bottom edge $q_2 = \frac{\delta T_2}{\delta x}.\frac{1}{t}$.

With uniformly-distributed load the forces T vary as a parabola along x, and since it is common to calculate the maximum values of T, that is T_c, then at any point $T = 4T_c\frac{x}{L}\left(1 - \frac{x}{L}\right)$ where x is measured from the support. Therefore $\frac{\delta T}{\delta x} = \frac{4T_c}{L}\left(1 - \frac{2x}{L}\right)$.

The bending moment is $M = (T_1 + T_2)\frac{d}{2}$, and $\frac{\delta M}{\delta x} = (q_1 + q_2)\frac{d}{2}$.

The axial thrust is $T_2 - T_1$ and the change of axial thrust along δx is

$$\frac{\delta T_2}{\delta x} - \frac{\delta T_1}{\delta x}.$$

From *Fig.* 3, equating the shearing stress (assumed to be constant along δx) to the unbalanced forces,

$$q.\delta x.t + q_1.\delta x.t = t\int_y^{y_1} \delta f.dy - \frac{(\delta T_2 - \delta T_1)}{td}(y_1 - y)t,$$

and, since $\quad f = \frac{My}{I}, \qquad q + q_1 = \frac{\delta M}{I\delta x}\int_y^{y_1} y.dy - \frac{1}{td}\left(\frac{\delta T_2}{\delta x} - \frac{\delta T_1}{\delta x}\right)(y_1 - y),$

and substituting for $\frac{\delta T_1}{\delta x}$, $\frac{\delta T_2}{\delta x}$, and $\frac{\delta M}{\delta x}$ as in the foregoing and integrating,

$$q = -q_1 + \frac{(q_1 + q_2)td}{2I}\left[\frac{y^2}{2}\right]_y^{y_1} - \frac{(q_2 - q_1)(y_1 - y)}{d}.$$

The value of q at the axis, where $y_1 = \frac{d}{2}$, $y = 0$, and $I = \frac{td^3}{12}$,

$$q_c = -q_1 + \tfrac{3}{4}(q_1 + q_2) - \tfrac{1}{2}(q_2 - q_1) = \tfrac{1}{4}(q_1 + q_2).$$

Combining with the first part,

$$q_m = \frac{3S}{2a} - \tfrac{1}{4}(q_1 + q_2) \qquad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

The form of the final distribution is as in *Fig.* 4 (*a*).

The proof of equation (6) given by Professor G. Winter and Mr. M. Pei is based on the area under the curve of the shearing stress multiplied by t and is equal to the vertical shearing force, that is by Simpson's rule

$$S = \left[\frac{d}{6}(q_1 + q_2 + 4q_m)\right]t \quad . \quad . \quad . \quad . \quad . \quad (7)$$

from which equation (6) follows.

Although Professor Winter and Mr. Pei state in their paper that q_m is the maximum shearing stress, it is not always true that the greatest shearing stress occurs at the middle of the slab. For example, if the forces T at the top and bottom of the slab are of opposite signs, the distribution of the shearing stress may be as in *Fig.* 4 (*b*), and the curvature may be inward. The actual distribution can be determined from equation (7) if q_1 and q_2 are given their appropriate signs.

For a slab in tension, such as a suspended tie slab with $q_2 = 0$ and with the reinforcement spaced out and stressed at m times the stress in the surrounding

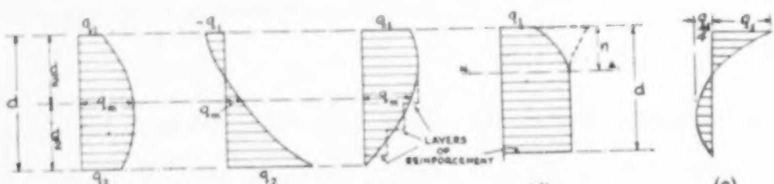

Fig. 4.

concrete, the member will be almost homogeneous. The shearing stresses will be partly transferred by bond at each layer of reinforcement, giving a distribution as shown by the broken lines in *Fig.* 4 (*c*). The approximate distribution is then that indicated by the full line in *Fig.* 4 (*c*), which is calculated from equation (7) with $q = 0$. If the reinforcement is concentrated near the bottom of the slab, the concrete is assumed to have cracked when the stress in the reinforcement is about the permissible tensile stress, and for this condition the shearing stress is constant below the neutral axis, and is transferred by bond to the bars. The distribution of shearing stress is then as in *Fig.* 4 (*d*), and

$$S = [q_m(d - n) + q_1 n + \tfrac{2}{3}(q_m - q_1)n]t$$

from which, as given by Professor Winter and Mr. Pei,

$$q_m = \frac{S}{t\left(d - \frac{n}{3}\right)} - \frac{q_1 n}{3\left(d - \frac{n}{3}\right)}.$$

In this case d is the depth to the reinforcement.

If the force T_1 is large, the distribution of shearing stress may be as shown by the broken line in *Fig.* 4(*d*).

The shearing force on a supported tie-slab is equal to zero, but the shearing

stress imposed by the force T is as shown in *Fig.* 4 (*e*), where q_m is $-\frac{q_1}{4}$, and the total area under the curve is equal to zero, and

$$q_1 = \frac{4T_c}{tL}\left(1 - \frac{2x}{L}\right).$$

Multiple-Span Structures.

MR. PEI, in a further letter, writes :—

In this journal for January, 1949, Mr. Ashdown presents an article on multiple-span prismatic thin-slab structures, the analysis in which, in my opinion, is incorrect. There are two types of prismatic structures, namely, statically-determinate and statically-indeterminate structures. The stresses in determinate structures can be found without reference to deformation but stresses in indeterminate prismatic structures can be determined only by considering the deflections of the joints. In statically-determinate prismatic structures, two slabs only meet

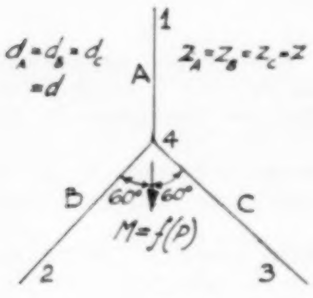

Fig. 5.

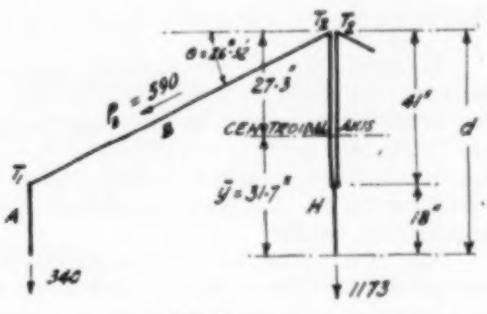

Fig. 6.

at any joint as in the examples given in Mr. Ashdown's *Fig.* 1. In indeterminate structures, three or more slabs may meet at a joint as in Mr. Ashdown's *Fig.* 3. The number of spans is not a criterion as both types can be single-span or multiple-span structures. Mr. Ashdown's numerical example deals with a statically-indeterminate structure.

For the solution of a statically-indeterminate structure in which three members meet at a joint, a deflection equation and two static equations are necessary, and are sufficient to determine the forces in each member. The deflection equation cannot be ignored. Similarly, if n slabs meet at one joint, in addition to two static equations, $n - 2$ deflection equations must be satisfied. The analysis comprises three types of equations : (*a*) Static equations expressing the equilibrium of the forces ; (*b*) Deflection equations expressing the compatibility of the deflections at the joint ; and (*c*) Stress equations expressing the equality of the edge stresses at the joint. The procedure in the article referred to includes the static and stress equations, but overlooks the deflection equations. As a result there are more unknowns than equations, to overcome which the forces in some slabs are arbitrarily assumed to be zero. The stresses computed by this procedure are incorrect for two reasons : (i) The analysis does not satisfy

the theorem of minimum strain energy; (ii) The deflections of the slabs are not compatible at the joint. A simple structure (*Fig.* 5) and the example given by Mr. Ashdown have been calculated by rigorous analysis by the writer. As the stresses deviate so much, the writer seriously doubts the validity of the proposed procedure.

EXAMPLE No. 1.—*Fig.* 5 shows a three-slab single-joint thin-slab prismatic structure. The slabs have the same dimensions and the angles between the slabs are 60 deg. By the proposed procedure, the stresses at the edges of the slabs are $f_1 = +\frac{7}{16}.\frac{M}{Z}, f_2 = f_3 = +\frac{1}{8}.\frac{M}{Z}$, and $f_4 = -\frac{1}{4}.\frac{M}{Z}$, indicating that slabs B and C bend upward under a downward load, which is obviously untrue.

The true stresses can be obtained by considering the entire structure as a simple beam, since it can be proved that for single-joint prismatic structures the thin-slab theory of prismatic structures and the theory of beams give the same stresses, which in this case are $f_1 = +\frac{7}{37}\frac{M}{Z}$, $f_2 = f_3 = -\frac{5}{37}\frac{M}{Z}$, and $f_4 = -\frac{1}{37}\frac{M}{Z}$.

EXAMPLE No. 2.—The example given by Mr. Ashdown is a symmetrical structure subjected to symmetrical loading. If this structure is analysed by two other methods as a statically-indeterminate structure the stresses in lb. per square inch are $f_0 = -956(-984)$, $f_1 = +216(+268)$, $f_2 = +90(-55{\cdot}5)$, $f_3 = -94(+268)$, and $f_4 = -173(-984)$. The stresses in brackets are those calculated by Mr. Ashdown's procedure; positive stresses are compressive and negative are tensile.

The rigorous analysis of a statically-indeterminate prismatic structure is intrinsically so difficult that E. Gruber in 1935 gave his solution in the form of a series of simultaneous non-linear differential equations.

MR. ASHDOWN replies:

I am obliged to Mr. Minglung Pei for pointing out the indeterminacy of the interior slabs, and the arbitrary distribution of load assumed on them. The intermediate tie-slab D is not necessary for stability, but serves to concentrate the tensile stresses (and therefore the reinforcement) below the roof slabs and reduces the risk of cracking due to the large tensile stresses which would be induced in the sloping slabs if the tie were omitted. Without the tie D the structure would be statically-determinate and the proposed procedure would be applicable. It is, however, applied to the more complex structure as an approximate method, and being approximate the calculated stresses should be greater than those obtained by a more rigorous mathematical analysis. Although the stresses obtained by Mr. Pei and myself are sensibly the same for the outer tie-slab and slopes, it must be admitted that the difference between the stresses in the intermediate tie calculated by the two analyses is greater than should be acceptable when comparing an approximate method with a more rigorous analysis. On the other hand, the validity of rigorous calculations of deformation based on homogeneous materials must be considered when dealing with a heavily-reinforced member (such as tie D) which must in part be assumed to be cracked.

Mr. Pei does not give his calculation for the stresses, but the following calcula-

tions, based on the slope of the roof slabs being 26 deg. 34 min., give longitudinal stresses of much the same value as those obtained by Mr. Pei.

As the slabs C, D, and E must deflect together, and since the stresses at the junction of C, D, and E must be the same in each member, it follows that, since the vertical deflection is proportional to $\frac{f}{y}$, y must be the same vertical distance from the junction for each slab C, D, and E. The present problem is simplified by the symmetry of the load and shape, and further simplification is possible by considering the slabs C, D, and E as one vertical slab H acted upon by $2T_2$ at the top, and by neglecting the force T at their junction (*Fig.* 6). First calculate $\bar{y}$, the position of the centroidal axis of H, and the moment of inertia I_H. The neutral axis will not be at $\bar{y}$, since the distribution of the stresses is modified by the imposition of the forces T_2 at the top.

$$\bar{y} = \frac{(600 \times 38{\cdot}5) + (180 \times 9)}{780} = \frac{23{,}100}{780} = 31{\cdot}7 \text{ in.}$$

$$I_H = \frac{180 \times 18^2}{12} + \frac{600 \times 41^3}{12} + (180 \times 22{\cdot}7^2) + (600 \times 6{\cdot}8^2) = 209{,}406 \text{ in.}^4$$

Therefore $d - \bar{y} = 27{\cdot}3$ in., $\frac{d - \bar{y}}{I_H} = 0{\cdot}00013$, and $\frac{\bar{y}}{I_H} = 0{\cdot}000151$.

The fundamental formula for joint AB is

$$2T_1\left(\frac{1}{a_A} + \frac{1}{a_B}\right) + \frac{T_2}{a_B} = \frac{1}{2}\left(\frac{M_A}{Z_A} + \frac{M_B}{Z_B}\right) \quad . \quad . \quad . \quad (a)$$

The longitudinal stresses at joint BC are

$$f_{BC} = \frac{T_1 - T_2}{a_B} - \frac{3(T_2 + T_1)}{a_B} + \frac{M_B}{Z_B} = \frac{-2T_1 - 4T_2}{a_B} + \frac{M_B}{Z_B}.$$

$$f_{BC} = f_{HO} = \frac{2T_2}{a_H} + \frac{2T_2(d - \bar{y})^2}{I_H} - \frac{M_H(d - \bar{y})}{I_H}.$$

Equating and transposing, for joint BC,

$$\frac{T_1}{a_B} + T_2\left\{\frac{2}{a_B} + \frac{1}{a_H} + \frac{(d - \bar{y})^2}{I_H}\right\} = \frac{1}{2}\left\{\frac{M_B}{Z_B} + \frac{M_H(d - \bar{y})}{I_H}\right\} \quad . \quad (b)$$

Also
$$f_{OH} = \frac{2T_2}{a_H} - \frac{2T_2\bar{y}(d - \bar{y})}{I_H} + \frac{M_H\bar{y}}{I_H}.$$

The total vertical load on the slabs C, D and E is $493 + 680 = 1173$ lb. per foot, and M_H is $-1173 \times 30^2 \times \frac{12}{8} = -1{,}583{,}550$ in.-lb. Therefore, in lb. per square inch, $\frac{M_H(d - \bar{y})}{I_H} = -206{\cdot}5$, $\frac{M_H\bar{y}}{I_H} = -239{\cdot}7$, $\frac{M_A}{Z_A} = 1700$, and $\frac{M_B}{Z_B} = 159$.

Evaluating (a) and (b),

$$0{\cdot}028\dot{8}T_1 + 0{\cdot}003\dot{3}T_2 = 929{\cdot}5$$
$$0{\cdot}003\dot{3}T_1 + (0{\cdot}006\dot{6} + 0{\cdot}001282 + 0{\cdot}00356)T_2 = -23{\cdot}75$$

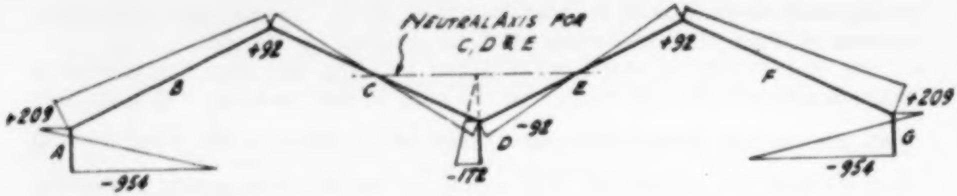

Fig. 7.

from which $T_1 = 33{,}550$ lb. and $T_2 = -11{,}790$ lb. The stresses in lb. per square inch are, therefore, as follows:

$$f_{AO} = \frac{33{,}550 \times 2}{90} - 1700 = -954.$$

$$f_{OA} = \frac{-33{,}550 \times 4}{90} + 1700 = 209.$$

$$\text{Check}: \; f_{BA} = \frac{(33{,}550 \times 4) - (11{,}790 \times 2)}{300} - 159 = 209.$$

$$f_{AB} = \frac{(-33{,}550 \times 2) + (11{,}790 \times 4)}{300} + 159 = 92.$$

$$\text{Check}: \; f_{HB} = \frac{-23{,}580}{780} - (23{,}580 \times 0{\cdot}00356) + 206{\cdot}5 = 92.$$

$$f_{OH} = \frac{-23{,}580}{780} + (23{,}580 \times 0{\cdot}004133) - 239{\cdot}7 = -172.$$

The longitudinal stress at the junction of C, D, and E is $92 - \frac{41}{59}(92 + 172) = -92$ lb. per square inch. The longitudinal stresses calculated in the foregoing are given in *Fig.* 7 and are almost identical with those calculated by Mr. Pei.

British Standard Code of Practice for Drainage.

A British Standard Code of Practice for "Building Drainage" (CP. 301, 1950. Price 7s. from the British Standards Institution.) contains recommendations for the planning of drainage schemes, and the selection, construction, and inspection of pipes and accessories for surface-water drains and sewers. The section on excavation is illustrated with drawings of timbering for trenches, shafts, and headings. The Code includes recommendations for cast-in-situ and precast concrete manholes.

Concrete pipes with spigot-and-socket joints are suitable for sewers of more than 6 in. diameter but are not recommended if the effluent is acid or if the pipes are to be laid in soils deleterious to concrete. An economy when using concrete pipes results from the fewer joints compared with glazed ware owing to the greater lengths of standard pipes. Concrete pipes with ogee or rebated joints are suitable for surface-water drains of all sizes.

Concrete used for the minimum amount of support and protection of pipes should be not leaner than 1 : 2½ : 5, but 1 : 4 : 8 or 1 : 10 may be used for protection additional to the minimum requirements or filling such as packing in headings or filling of soft patches in the bottom of a trench. Recommendations for the mixing and placing of concrete are given. If concrete pipes are laid less than 4 ft. or more than 20 ft. below the surface they should be entirely surrounded by concrete. If from 4 ft. to 20 ft., they need only be surrounded if laid in a heading or under buildings; otherwise they need only be haunched, or house drains at a distance from a building need only be bedded.

Residential Flats for the London County Council.

MONOLITHIC REINFORCED CONCRETE CONSTRUCTION.

AMONG the many blocks of flats being erected for the London County Council, some are of monolithic reinforced concrete construction. These include some of the blocks at Woodberry Down, Stoke Newington, and Queen Caroline Street, Hammersmith, and all the blocks at Flower House estate, Lewisham. The reinforced concrete work and related structural details of the blocks on these three sites are designed by, or on behalf of, the contractors and are based on the plans and requirements of the Housing and Valuation Department of the London County Council under the direction of Mr. Cyril H. Walker, O.B.E., M.C., F.R.I.C.S., F.R.I.B.A., M.I.Mun.E., and the Housing Architect, Mr. Sydney Howard, L.R.I.B.A.

The superimposed load for which the floors and flat roofs are designed is 50 lb. per square foot, and for the public balconies and stairs 100 lb. per square foot. For the tall blocks at Woodberry Down resistance to wind pressure is included. The reinforced concrete is designed in accordance with the by-laws of the London County Council, that is the maximum tensile stress in mild steel reinforcement is 18,000 lb. per square inch and the maximum compressive stress in 1 : 2 : 4 concrete is 950 lb. per square inch in bending and 760 lb. per square inch in direct compression. The modular ratio is assumed to be 15.

In the following the buildings at Lewisham and Hammersmith are described, and it is intended to describe those at Stoke Newington in a later number.

Fig. 1.

Flower House Estate, Lewisham.

There are fifteen blocks comprising 330 flats at the Flower House estate (*Fig.* 1). All the structures are of reinforced concrete, the general design of which is shown in *Fig.* 2. *Figs.* 3 to 7 show the structures in course of construction. Five of the structures are of four stories and are each 252 ft. 7½ in. long and 37 ft. high to the ceiling of the top story. The remaining ten blocks are of three stories and are 157 ft. 4 in. long and 27 ft. 10 in. high. Each block is 24 ft. 6 in. wide. Con-

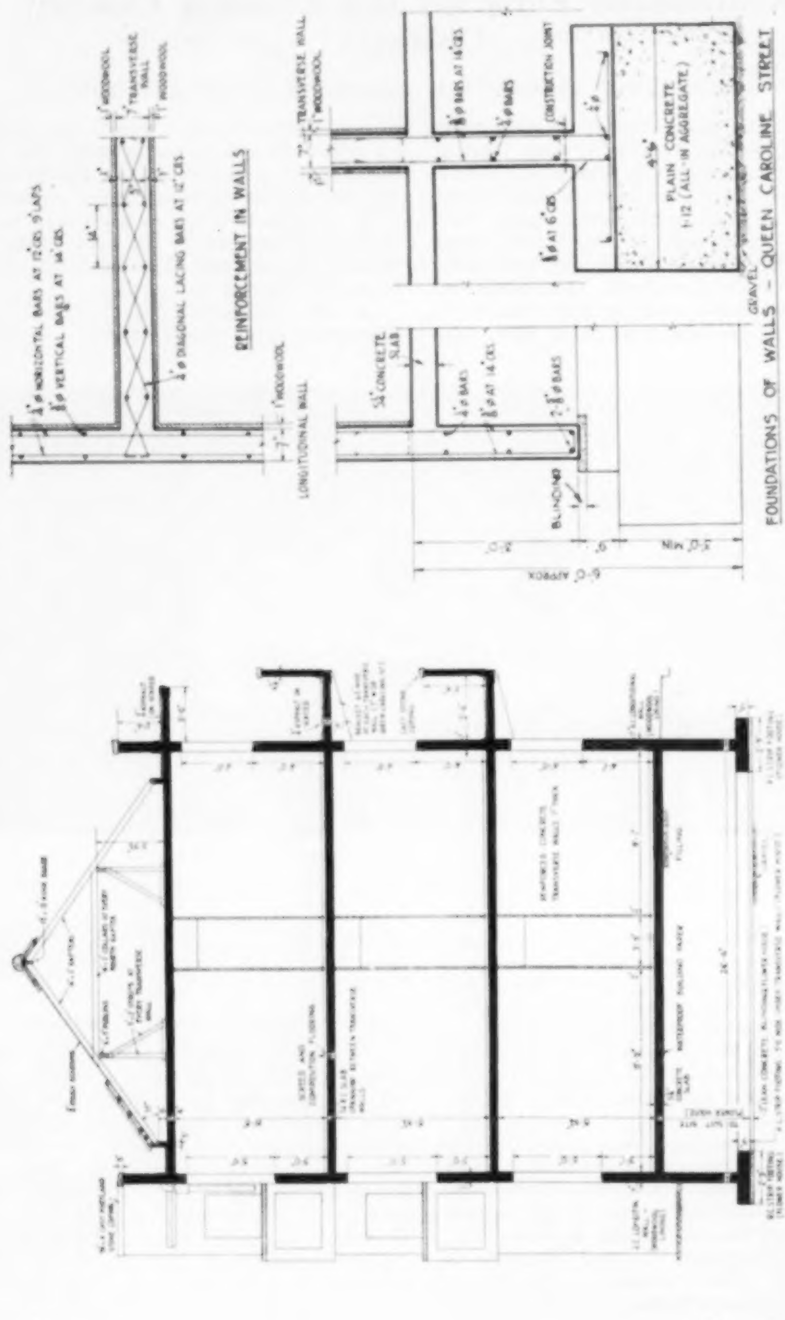
REINFORCEMENT IN WALLS
TRANSVERSE WALL
LONGITUDINAL WALL
CONSTRUCTION JOINT
PLAIN CONCRETE
1:12 (ALL-IN AGGREGATE)
GRAVEL
BLINDING
FOUNDATIONS OF WALLS – QUEEN CAROLINE STREET

Fig. 2.

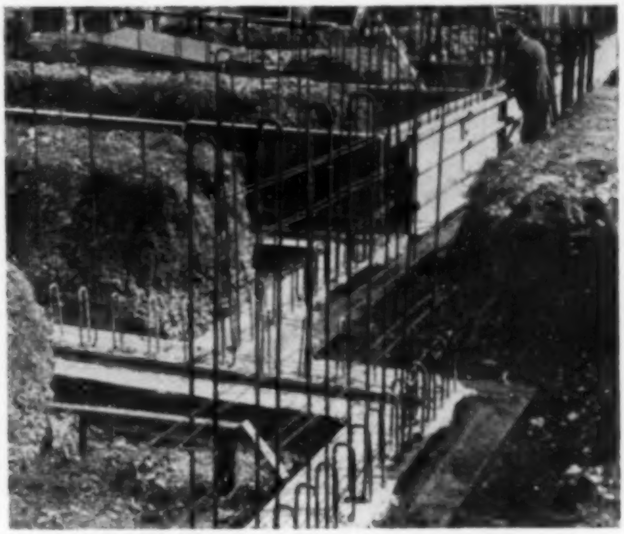

Fig. 3.—Wall Footings.

struction started in February, 1947, and the flats were completed in 1949.

The flats are planned to suit monolithic cast-in-situ reinforced concrete construction, the main load-carrying members being the transverse walls comprising 7 in. of reinforced concrete faced on both sides with 1-in. wood-wool slabs, the exposed face of which is plastered. The external longitudinal and end walls are constructed similarly to the internal transverse walls but with the wood-wool lining on the inside face only and an applied finish on the outside. The reinforcement in the transverse walls comprises crimped horizontal bars, as in *Fig.* 2, which hold the vertical bars rigidly in position. Partition walls are of 2-in. breeze blocks.

The foundations, which are generally on gravel, are reinforced concrete strip footings not less than 3 ft. wide and 9 in. thick, but the dimensions and depth depend on the nature of the ground. For one block on weak soil, a reinforced concrete raft is provided. The ground floor comprises a 5½-in. reinforced concrete slab designed as a suspended floor but laid on filling, covered with waterproof building paper and finished generally with composition flooring. The upper floors are

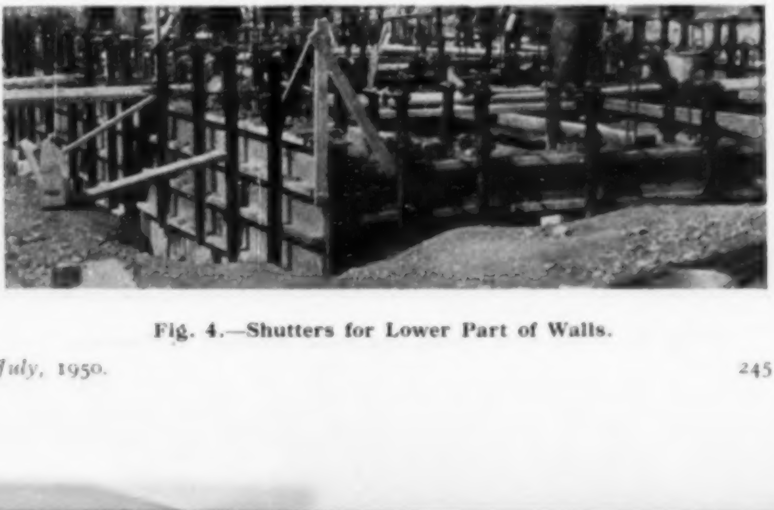

Fig. 4.—Shutters for Lower Part of Walls.

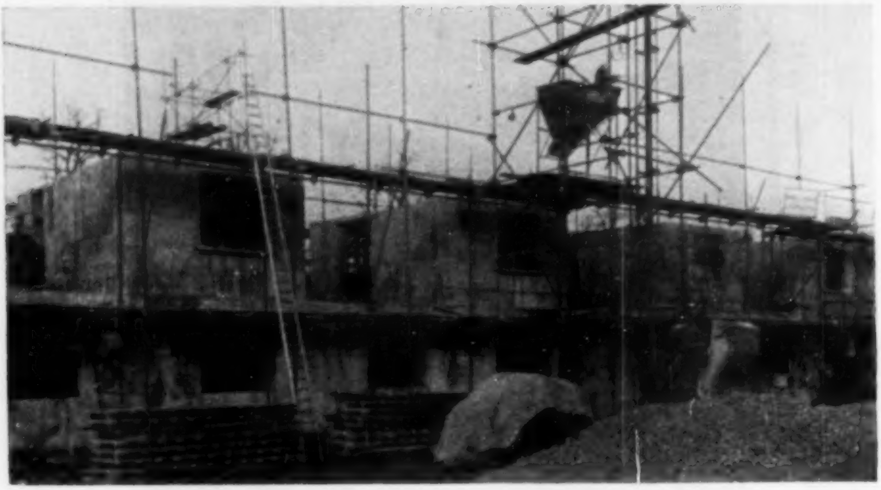

Fig. 5.—Construction of Second Story.

also reinforced concrete slabs 5½ in. thick and all floors span between the transverse walls. The transverse distance between the faces of the 7-in. longitudinal reinforced concrete walls is 23 ft. 4 in. The story-height is 9 ft. 1½ in. The roofs are pitched and comprise a 4-in. flat reinforced concrete ceiling slab supporting timber rafters, the covering being double Roman-pattern Bridgwater sand-faced tiles.

The balconies comprise 5½-in. reinforced concrete slabs supported on cantilevers that occur at each transverse wall. The balcony walls are 4½ in. thick and are surmounted by precast concrete copings, as are also the parapets at the roof.

Fig. 3 shows part of the footings, the

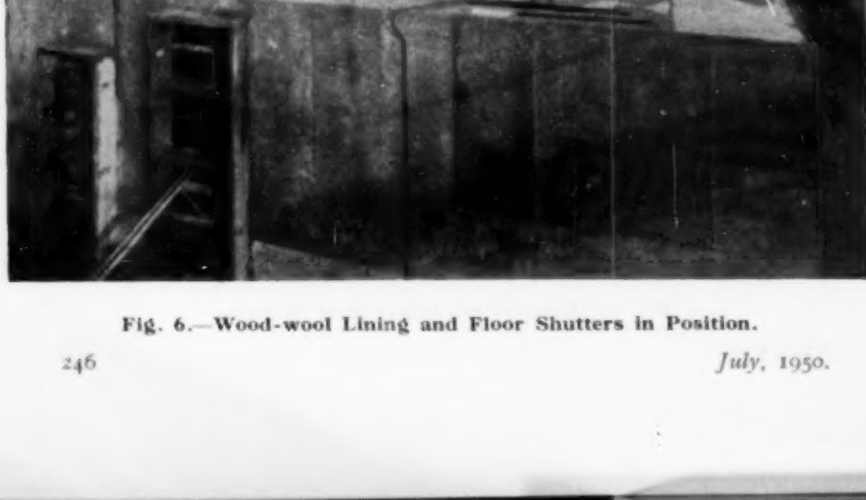

Fig. 6.—Wood-wool Lining and Floor Shutters in Position.

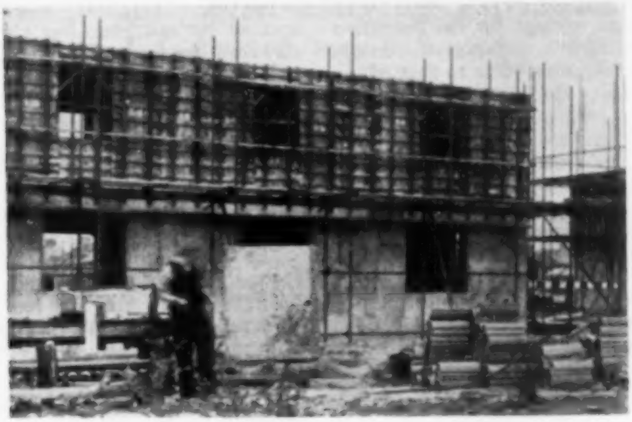

Fig. 7.—Shuttering for Second Story.

reinforcement fixed for the walls, and the ¾-in. plywood shuttering for the walls up to ground-floor level being erected. The panels of shuttering were maintained in position by steel clamps (*Fig.* 4), the distance apart of which was adjusted by two cross screws between the heads of each pair of soldiers. The walls were constructed up to the level of the ground floor, the shuttering removed, the filling placed and consolidated, the building-paper laid, and the ground-floor slab cast. The shuttering was then erected for the walls of the bottom story and the first floor. The walls were cast in one lift from floor to floor, consolidation by vibration ensuring compaction of the concrete under the windows. The concrete was raised to the level of each floor by a hoist in a steel-scaffold tower (*Fig.* 5) and transported to the position of placing in wheelbarrows fed from a hopper at the top of the hoist. The method of constructing each story is shown in *Fig.* 6 where are seen in place the metal-faced plywood shutters for the soffit of a floor slab of one cell, the wood-wool lining for one face of each transverse wall, the timber frames for door-openings, and the electric conduits. This assembly was supported on a light steel framework which was erected first in alternate cells, leaving the intermediate cells as a working space from which the reinforcement in the walls and the wood-wool slabs, acting as shuttering for the other face of the walls, were erected. The corresponding steel framework was then erected in the intermediate cells, the shuttering for the floor slab over these cells fixed, and when the shuttering for the outside of the longitudinal walls (*Fig.* 7) was erected, concreting of the walls and floor of one story proceeded.

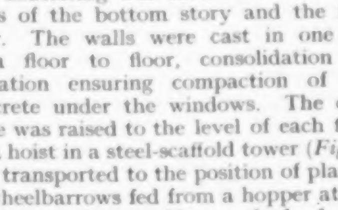

Fig. 8.

The internal finish to the walls and ceilings is distemper, the dadoes being in plastic paint. To the outside face of the walls a pink or buff textural cement finish was applied by a machine, the wall below the level of the window sills of the bottom story being rubbed to give a contrasting finish. The walls of stair-wells are finished in terrazzo. The stairs and half-landings are cast in situ. The cost of the blocks of flats was about £467,000; the total cost of this estate is estimated to be about £600,000.

Queen Caroline Street, Hammersmith.

There are five almost identical blocks at Queen Caroline Street, Hammersmith,

comprising 105 flats. Each block is of five stories and, except for the provision of eaves instead of a parapet, the design and construction are similar to that of the blocks at Flower House estate. Four of the blocks are each 120 ft. 6 in. long, and one is 145 ft. long. Each block is 24 ft. 6 in. wide and the height to the eaves of the pitched roof is 46 ft. Construction commenced in March 1949 and completion is expected during 1950. The average rate of progress of the constructional work in good weather is one story in about three weeks. The estimated cost of the five blocks is about £186,000. The external finish to the concrete walls is to be a textural cement rendering applied by machine, but three different coloured cements will be used so that the bottom story and top stories will contrast in colour with the intermediate stories. The aggregates are washed broken gravel from pits in the Thames valley and washed pit sand. *Fig.* 8 shows a cross section of the reinforced concrete columns in the bottom story supporting the balcony adjacent to the lift shaft.

The contractors for the flats at the Flower House estate and at Queen Caroline Street are the Kent & Sussex Contractors, Ltd. The reinforced concrete was designed by Mr. W. C. Andrews.

Book Reviews.

"Handbook of Rigging." By W. E. Rossnagel. (London: McGraw-Hill Publishing Co., Ltd. 1950. Price 40s. 6d.)

ALTHOUGH dealing mainly with American practice regarding cranes and scaffolding used in constructional and industrial operations, there are many data of general use dealing with ropes, chains, hooks, and manual and mechanical lifting devices of all kinds. The inclusion of chapters on elementary mechanics, the properties of timber, the prevention of accidents, first-aid, and other matters on the border-line of the subject, makes the book of considerable value to those for whom it is intended, that is scaffolders, crane-drivers, maintenance engineers, and supervisors of constructional work. It should, however, be equally useful to designers who may lack experience of the practical erection of structures. The book, which contains about 300 pages, is written so far as possible in not too-technical language, but many terms known to the trades concerned are necessarily used, and generally explained, although in some cases other terms may be more common in this country.

"Soil Survey Procedure." (London: H.M. Stationery Office. 1949. Price 1s.; 30 cents in U.S.A.)

THIS booklet of about 40 pages is Road Research Technical Paper No. 15 of the Department of Scientific and Industrial Research, and is a reprint, with additional matter, of Road Research Bulletin No. 4 (1946). The descriptions of the equipment and procedure for conducting soil-surveys of sites of roads, classifying soils, investigating ground water, and allied matters are well illustrated and accompanied by tables. Soil classification is based on that of the U.S. Corps of Engineers. Reference is made to the tests of the British Standards Institution (B.S. No. 1377, 1948; reviewed in this journal for March 1949, page 103). The authors have kept to the fore the practical side of the subject, and use of the term "soils mechanics" is rare. That they are aware that they are dealing with old principles is evident from the quotations given from Parnell's "Treatise on Roads" (1833), based on Telford's practice: "A vertical section should be made, and the nature of the soil or different strata should be shown over which each apparently favourable line passes, to be ascertained by boring; for it is by this means alone that the slopes at which the cuttings and embankments will stand can be determined and calculated," and "If bogs or morasses are to be passed over, the depth of the peat should be ascertained by boring; and the general inclination of the country for drainage should be marked."

"Contractors' Plant." By H. O. Parrack. (London: Sir Isaac Pitman & Sons, Ltd. 1950. Price 25s.)

SINCE the equipment used by civil engineering contractors is dealt with from the points of view of organisation, operation, and maintenance, much of this book concerns the care and repair of machines and for this reason should be of value to civil engineers who have to control the use of plant but have not much experience of mechanical engineering. The administration of the plant department of contracting firms and the training of operators and mechanics are also considered.

Construction with Moving Forms.—V.*

By L. E. HUNTER, M.Sc., A.M.Inst.C.E.

Plant and Progress.

WITHOUT attention to details, moving-form construction will be fraught with difficulties and stops may occur during the course of the work.

Materials.—All quantities of materials must be accurately calculated and the amounts required on the site weighed, allowing a margin in case the supply should fail while the work is in progress or be reduced due to circumstances beyond the contractor's control. Not less than half the required quantities should be on the site at the start of the work, but the remainder must be there several days before it is required. For small structures, and for large structures if sufficient space is available, all materials should be on the site before moving

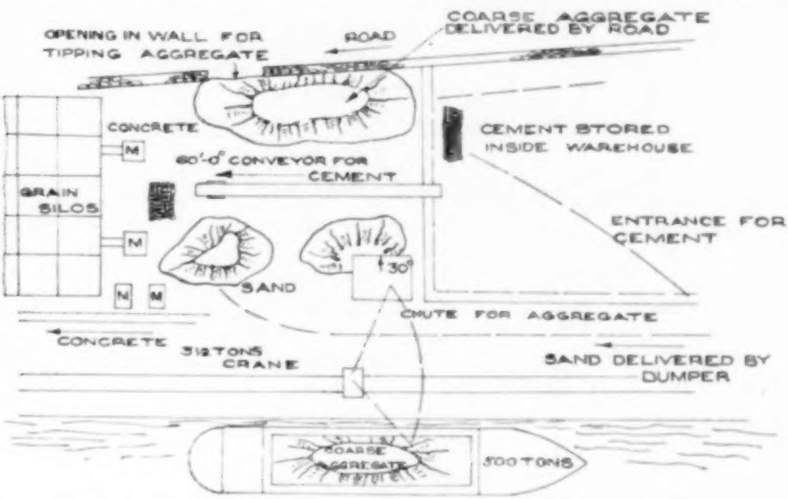

Fig. 30.—Arrangement of Materials for Silo with 70 Bins.

of the forms commences and stored so as to reduce as much as possible the distance from the store to the work. *Fig.* 30 shows the arrangement of the stores and plant for a grain silo of more than 70 bins. In this case the disposition of the materials was severely hampered by the congestion of the site, and in order to obtain storage space for the cement the ground floor of an adjacent warehouse was used. This would have necessitated a haul of more than 150 ft. from the door of the warehouse, but an opening was made in the wall nearest to the work and, with the use of a large belt conveyor one end of which extended into the warehouse, the supply of cement was able to keep pace with the requirements.

Concrete Plant.—The lifting of the materials to the top of the work must be efficient, and allowance made for one or other of the lifting appliances breaking down. It is necessary to allow more than sufficient plant to give the necessary capacity. If mixers are used, then at least one extra machine should be provided.

* Continued from June number.

An extra hoist is also desirable because on a big structure the reinforcement alone may require the full capacity of one hoist.

It is debatable whether a large batching plant and one mixer or several smaller mixers or batchers should be used. The writer considers that the former is not so suitable for a structure large in plan because if a breakdown occurs of the batching plant the whole of the work stops. The cost of providing a similar large batching plant as a standby is prohibitive, and it is better to have several small machines around the building. This has the advantage that if one mixer fails the others continue and the work is not stopped, while if a spare mixer be available the work can continue with only a short break in the full capacity of the machines. If several mixers are used it may be necessary to haul the materials to the mixers farthest from the supplies, and for this purpose small dumpers,

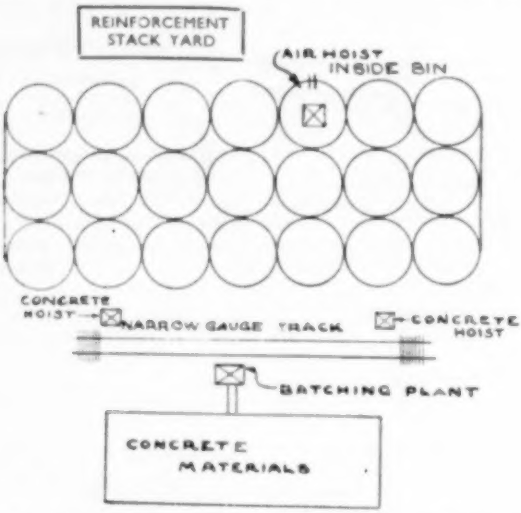

Fig. 31.—Arrangement with One Large Batching Plant and Mixer.

or skips running on narrow-gauge track would be suitable. On the other hand, if small mixers are disposed around the work at points to which the concrete can be readily taken by barrows over short distances from the heads of the hoists, the time lost on the ground in transporting the unmixed materials to separate mixers is more than regained by the short distance of travel of the barrows on the deck. Again, if one large batching plant is used, then, in order to deliver concrete to the various parts of the structure, narrow-gauge track and skips must also be used; otherwise, if hoists are used at one end, it is long and tiring work to push barrows to the far end of the structure. *Fig.* 31 illustrates the case of a site where one large batching plant was used, and *Fig.* 32 shows the use of several small machines.

If a central batching and mixing plant is used the capacity should be ½ cu. yd. to 1 cu. yd., depending on the size of the structure. Narrow-gauge track for the skips transporting concrete from the mixer to the hoists should not be too light, otherwise derailments may occur. A suitable weight of track is 20 lb. per foot,

with turntables. A side-tipping skip is useful, having two such on each line so that one is being filled while the other is taking a load from the batcher to the hoist. Dumpers, or diesel locomotives drawing a skip, are useful alternatives when large quantities of concrete have to be transported from a central-mixing plant to hoists at the ends of long structures.

Concrete pumps have been used and are satisfactory if the greatest lift does not exceed 135 ft. with a new pump, or 110 ft. with a well-worn pump. Experience shows that the best method of using pumps with moving forms is for the pipe to discharge into a hopper on the deck. To move the end of the pipe about the deck is not satisfactory. If hoppers are not provided, several branches of the pipe should be provided, each branch delivering to a different part of the structure. This method reduces the amount of barrowing and the number of men required on the deck, but it is untidy and often results in much concrete being spilled.

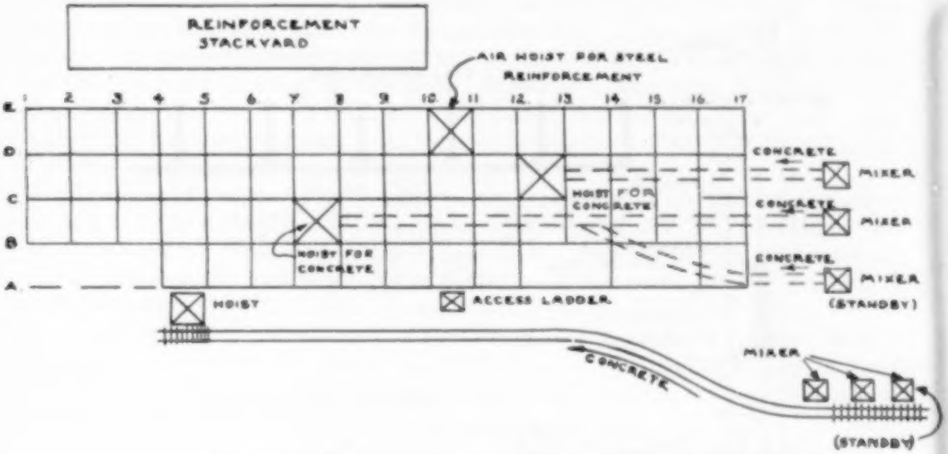

Fig. 32.—Arrangement with Several Small Mixers.

The choice of the best type of hoist is easily settled on the site, and it is customary to use an electric or diesel-engine drive for hoisting the concrete. One hoist for each 20 cu. yd. of concrete to be placed in an hour is generally required. The hoists used for the concrete should not be used for reinforcement or other materials. Workmen should not be permitted to ride on the materials' hoists, and in the case of a tall structure (such as a chimney or tall silo) a passenger hoist might be necessary. This hoist could be occasionally used for light equipment.

Reinforcement.—For hoisting the reinforcement a simple type of electric winch or air hoist is often the most convenient. To permit free passage of the bars, the hopper-bottom of one of the bins may be omitted and the bin used as a lift-well. Reinforcement may be lifted with a gin-wheel up to 70 ft., but this method requires more labour. The reinforcement should not be taken to the deck in small quantities ; plenty of bars should always be at hand on the deck, and racks provided for storing them. As the number of steel-fixers required while the forms are moving is considerably more than the whole reinforcement

gang before this process starts, it is essential to bend all the steel required before moving of the forms begins. The reinforcement should be stacked so that the least amount of sorting is necessary when the bars are required. As the reinforcement, after being bent, may lie for some time before it is used it may be necessary to remove mill-scale, rust, oil, or grease. It is essential that the bars be cleaned before moving of the forms starts, because if it is necessary to do so when the work is in progress the work may be delayed; also, care must be taken to protect the bars from becoming dirty again after having been cleaned. Where space permits the provision of a shelter is the best means of keeping the reinforcement clean.

Steel-fixers must be allocated to each part of the deck. The steel-fixing foreman should have a copy of the reinforcement schedules and be fully responsible for this work. The upward movement is too fast to allow each bar to be checked, but it is imperative to check the spacing of the bars, and this must be done quickly.

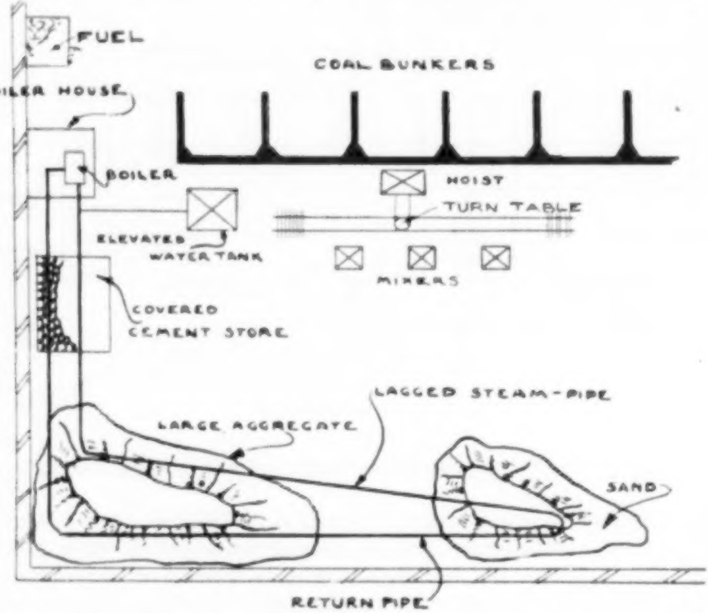

Fig. 33.—Arrangement for Heating Concrete Materials and Mixing Water.

Construction in Cold Weather.—As previously stated, it is essential to provide means of heating in periods of low temperature. The aggregate should be steam heated and, if possible, the cement stored in an enclosure heated possibly by a branch of the aggregate-heating system. It is an advantage for the mixing water to be warmed, and a separate tank with steam-pipes running through the water should be provided at a level suitable for discharge into the drum of the mixer. The tank should be enclosed and insulated with several thicknesses of sacks, boards, or brickwork. The heating of the aggregate with a steam-pipe is most effective when the pipe has several branches at different levels in the heap. A sound boiler of sufficient capacity is required, and in cold weather the steam-

heating plant should be in operation at least one day before the moving of the forms commences in order to overcome any effects of frost that may have occurred previously. *Fig.* 33 shows an arrangement of a temporary boiler plant and pipes for steam heating the aggregates and cement with a branch taken to a water tank. The temperature of the concrete as it leaves the mixer should not be less than 60 deg. F. for there to be no risk of ill effects of frost before placing. Whenever possible the top of the heap of aggregate should also be protected by tarpaulins.

Precautions must be taken to protect the placed concrete against effects of the sun, cold air, rain, and drying winds. In hot weather a hose-pipe should be available so that water can be sprayed upon the walls at frequent intervals. Cold winds are most likely to occur in the early morning, and it is advisable to have available several canvas walls which at short notice can be erected as wind-shields. They can easily be removed when the temperature rises.

Emergency Equipment.—It is essential to provide spare parts for the jacks, the most important being spare jaws in case the original jaws break. Extra jack-rods are necessary since, due to excessive load, a rod may buckle in the middle, or it may protrude from the wall at the end due to eccentricity of load on the tube at a junction of two rods. When this occurs the jack should be immediately relieved of all load and the buckled rod cut out of the wall. For a time this yoke must remain free of load until the part of the rod embedded in the wall is sufficiently long to take the load from the yoke.

It is also desirable to have at hand small shutters of plywood or boarding in case a large cavity occurs in a wall. A shutter should be fastened to the reinforcement on each side of the wall so as to cover the cavity, and concrete placed between the two shutters by buckets. The removal of the shutters is effected from a bosun's chair. Such a shutter will be illustrated later.

Labour and Progress.—The number of men required is at least double the number necessary for placing the same amount of concrete in normal fixed shuttering, as the following list of staff and men employed on a grain silo of seventy 15-ft. square bins shows.

	For Moving Forms	For Fixed Shutters
Agent	1	1
Engineers	2	1
Foremen	2	1
Charge-hands	5	2
Steel-fixer foreman	1	1
Steel-fixers	24	10
Labourers on steel	12	—
Scaffolders	8	4
Hoist drivers	4	1
Mixer drivers	4	1
Barrow gang (on deck)	14	2
,, ,, (at mixers)	36	6
Concrete finishers	14	2
Punners	30	4
Jack-men	30	—
Carpenters	3	10
Jack-rod changers	12	—
Spare labourers	12	6
Storeman	1	1
Timekeepers	3	2
Catering	6	4
Total	224	59

The foregoing is based on a silo the lower 30 ft. of the walls of which were constructed with fixed shutters and the upper 90 ft. with moving forms. In this example the comparison is not quite advantageous to moving-form construction as the work was done in single shifts only, stopping each night. In another case, for twelve bins of the same size, 160 men were required including two shifts; about 90 men would be required for a similar structure using fixed shutters.

Without considering the relative quantities of work done in the same time, a true comparison of speed of construction cannot be drawn between fixed shutters and moving forms. In the same time four times as much height of wall can be concreted with moving forms as with fixed shutters. Normally a minimum height of 5 ft. per ten-hour shift may be constructed, but in many cases, because the speed of the work increases considerably after the first day or two, 7 ft. of wall can be constructed although the rate depends on the simplicity of the reinforcement and the dimensions of the wall. To compare the usual rate of progress of a silo with fixed shutters and one built with moving forms it must be borne in mind that when using fixed shutters it may be necessary to build an outside scaffold the full height of the structure in addition to scaffolds in each bin, which may also have to be taken from a level near the ground. It is reasonable to expect the walls of a structure 100 ft. high to be completed in 10 to 14 consecutive days of 24 hours' continuous concreting. In addition there should be included some of the time during which the moving forms are being made. On the other hand, if fixed shutters were used, not only must time be allowed for erecting and removing scaffolding and making the panels of shuttering, but if a 3-ft. lift were completed every day it would take 34 working days to reach the top; if this number of days were doubled, it might still be considered to be good progress.

(*To be continued.*)

A Simple Test of the Consistency of Concrete.

A SIMPLE method of testing the consistency of concrete is described in a recent number of the Bulletin of the American Society for Testing Materials. The method is to observe the penetration into the surface of the concrete of a 30-lb. metal disk of 6 in. diameter. By a coincidence, the penetration equals roughly half the slump. The disk (*Fig.* 1) has a hemispherical base, to which is attached a handle passing through a light metal stirrup. The stirrup rests on the surface of the concrete into which the disk sinks, the amount of penetration to the nearest 0·1 in. being read on graduations on the handle. The mean of three readings taken at different parts of the surface of the concrete represents the consistency to be recorded. The test appears to be more sensitive and quicker than the slump test, and is useful in maintaining control of the consistency of the concrete as placed.

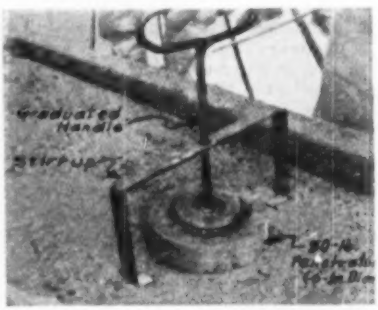

Fig. 1.

A Prestressed Concrete Road Bridge in Lancashire

A PRESTRESSED concrete road bridge of 80 ft. span (*Fig.* 1) was erected recently over the river Tame at Denton sewage works, near Manchester. The carriageway is 10 ft. wide (*Fig.* 2) and the structure is designed to carry 10-ton lorries. The soffit of the girders are at such a level that sufficient clearance is obtained at times of flood, and consequently the structural depth available is only 3 ft.

Fig. 1.

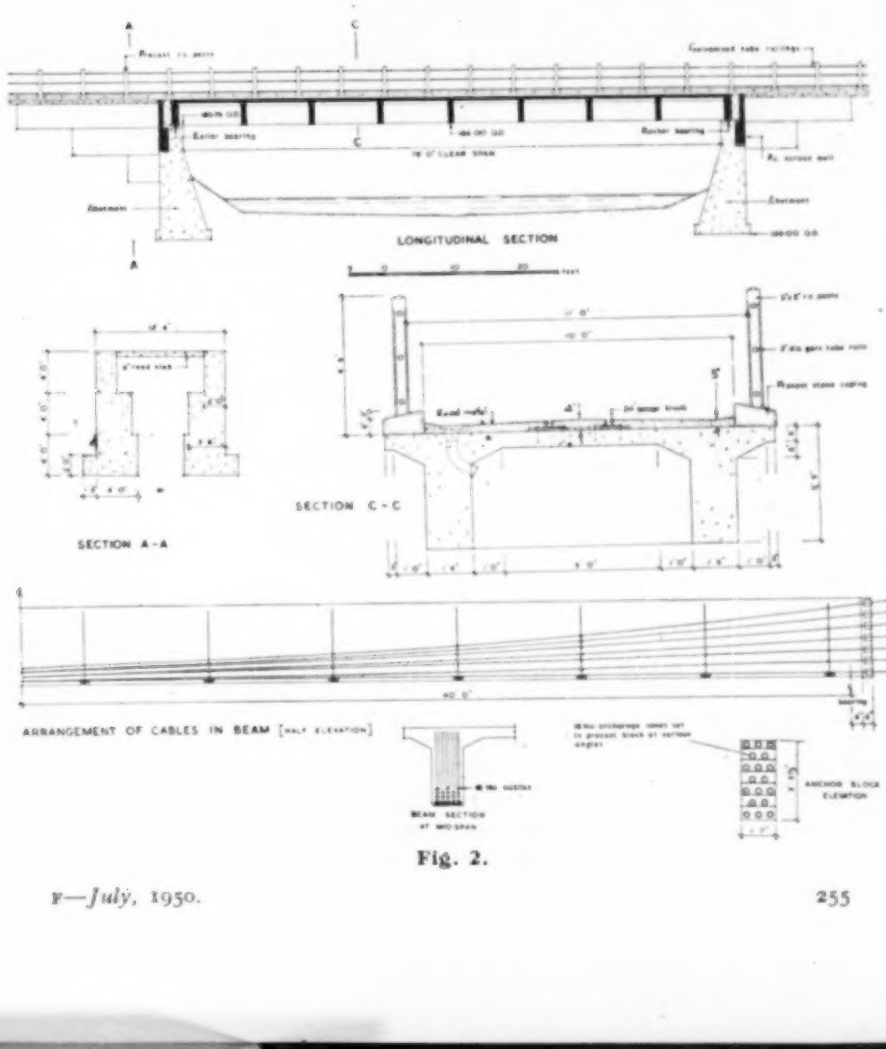

Fig. 2.

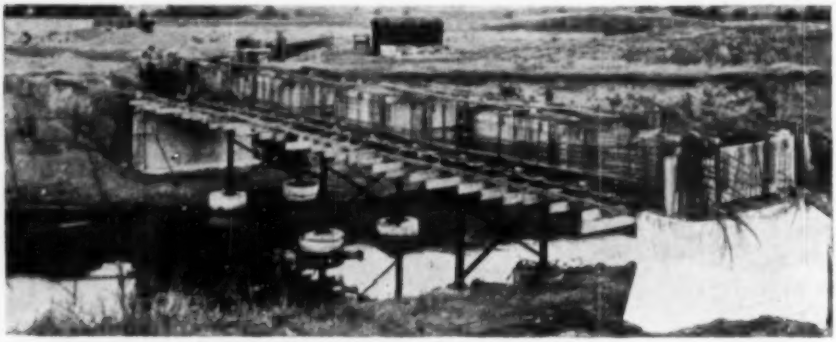

Fig. 3.

9 in., a restriction that made a design in prestressed concrete advantageous.

The structure, which was cast in situ on a temporary staging (*Fig.* 3), comprises two concrete beams, 18 in. wide and 3 ft. 9 in. deep, cast monolithically with a 6-in. road slab (*Fig.* 2). Each beam contains eighteen cables which were constructed and stretched in accordance with the Freyssinet system, and which induce in the concrete a state of stress such that no tensile stresses occur under the working loads. The specified cube-strength of the concrete was 6000 lb. per square inch at 28 days, but this strength was generally exceeded at seven days.

The beams were cast in 10-ft. sections, with vertical joints between each section. An advantage of casting the beams in sections was that the concrete in each section could be consolidated thoroughly by external vibrators without interfering with the setting of the concrete in adjoining sections. The cables were stretched by double-acting jacks when the concrete had matured. The stretching of the 36 cables required a total force of 900 tons and took about five days using two jacks. Under the prestressing the beams lifted about ⅜ in. above the shuttering at the middle of the span.

In the main beams there are about 36 cwt. of steel in the cables and 6 cwt. of mild steel provided to facilitate the fixing of the cables to a parabolic curve. The contract price for the erection of the bridge, abutments, river wall, and approaches was about £8000, of which £2300 was for the erection of the prestressed beams and deck slab. The joint consulting engineers are Messrs. G. B. Kershaw & Kaufman and Messrs. L. G. Mouchel & Partners, Ltd. The work was carried out to the general designs and under the supervision of the engineers and surveyors of the Denton Urban District Council, Mr. D. S. Graham, B.Sc., and Mr. J. B. Cooke. The contractors for the work were Messrs. R. & T. Howarth, Ltd.

Lectures on Road Materials and Construction.

Lecture courses on road materials and construction will be resumed at the Road Research Laboratory during the autumn and winter of 1950–51 as follows—Soil mechanics: October 10 to October 20 and December 12 to December 22. Concrete: October 24 to November 2 and January 2 to January 11. Tar and bituminous materials: November 7 to November 23 and January 16 to February 1. A fee of £7 7*s.* will be charged for each course. Forms of application can be obtained from the Road Research Laboratory, Harmondsworth, Middlesex.

The Reconstruction of East India Graving Dock, London.

In a recent number of "The Dock and Harbour Authority", Mr. F. W. Davis, M.Sc., M.I.C.E., describes the reconstruction of the war-damaged graving docks belonging to the London Graving Dock Co. and situated on the river Thames at Poplar. The East India graving dock was subjected to repairs amounting almost to reconstruction although, by the use of reinforced concrete, complete demolition and re-building of the side walls of the dock were avoided. This dock was originally an open slipway but was converted into a graving dock with timber side walls and floor, brick knuckles, a caisson at the entrance, and a circular brick head. About the year 1892 the timber floor was replaced by a vented plain concrete slab carried on the old timber piles. In 1903 the timber side-walls were replaced by mass concrete gravity walls. Other alterations resulted in the cross section of the dock being eventually as shown by broken lines in the cross section in *Fig.* 1.

The recent remedial works were designed so that the width of the dock was not reduced, and the floor was lowered 18 in. The strengthened walls are designed to resist the pressure due to a surcharge of 1 ton per square foot on ground at the back of the wall to within 14 ft. of the coping. The floor is designed to resist hydrostatic uplift without vents and to carry a load of 15 tons per foot on the keel blocks.

The dock has an overall length of 290 ft. inside the caisson, a width of 52 ft, at keel-block level, and a depth of 25 ft. below the coping at flood-prevention level (18·25 ft. O.D.). Freely water-bearing drift gravel extends to 2 ft. above the new floor and is overlain by Thames mud and made ground. The stability of the old walls depended upon tensile and high compressive stresses in the concrete, and the walls were therefore susceptible to damage from shocks, such as exploding bombs, as was evident by the spalling and cracking of the bilge altars and the floor. The method of stabilising the walls is shown in *Fig.* 1. The new reinforced concrete floor and sill are generally 3 ft. thick and are integral with a continuous haunch carried up 7 ft. above the floor. The haunch is extended by buttresses at 15 ft. centres which project 18 ft. above the floor, and are connected to new anchor tie-rods. The old walls are cut away for the haunches and buttresses to avoid any projection into the dock and to enable the width at keel-block level to be increased from 47 ft. to 52 ft. The reinforced concrete was designed to resist the varying conditions of hydrostatic uplift and the load on the keel-block combined with the bending and thrust from the side walls. The bending stresses in the haunches are partly relieved by the buttresses and tie-rods, and to a lesser extent by the weight of the wall. The arrangement of the reinforcement is also shown in *Fig.* 1. At the circular head the main bars in the floor are arranged similarly to those in the side walls but are placed radially.

The new floor and haunches were constructed in alternate transverse strips 8 ft. 6 in. and 6 ft. 6 in. wide, the narrower strips coinciding with an inset buttress. The old floor and haunches were cut out in corresponding strips to provide a continuous sequence of operations, that is demolition and excavation, fixing reinforcement, concreting, and curing. Two settings of struts across the dock shored the side walls in the vicinity of the demolition. Concrete mixed in the proportions of 1 : 2½ : 3½ is used for the floor and buttresses and 1 : 1½ : 3 for the sill and piles (*Fig.* 1). The average amount of reinforcement is about 8 lb. per cubic foot of concrete. The cover of concrete over the bars is not less than 3 in. Corrugated copper strips are embedded in the transverse joints of the floor, but were not entirely successful in preventing seepage as the strips were damaged when cutting out the adjacent section of the old floor. Concrete in each transverse strip was deposited in one operation for the full width of the floor.

The gravel foundation was dewatered by two transverse rows of well-points about 40 ft. and 100 ft. respectively from the caisson. The screened well-points, inserted in holes drilled through the floor and then water-jetted down to the required level, were put down to 6 ft. below the underside of the new invert,

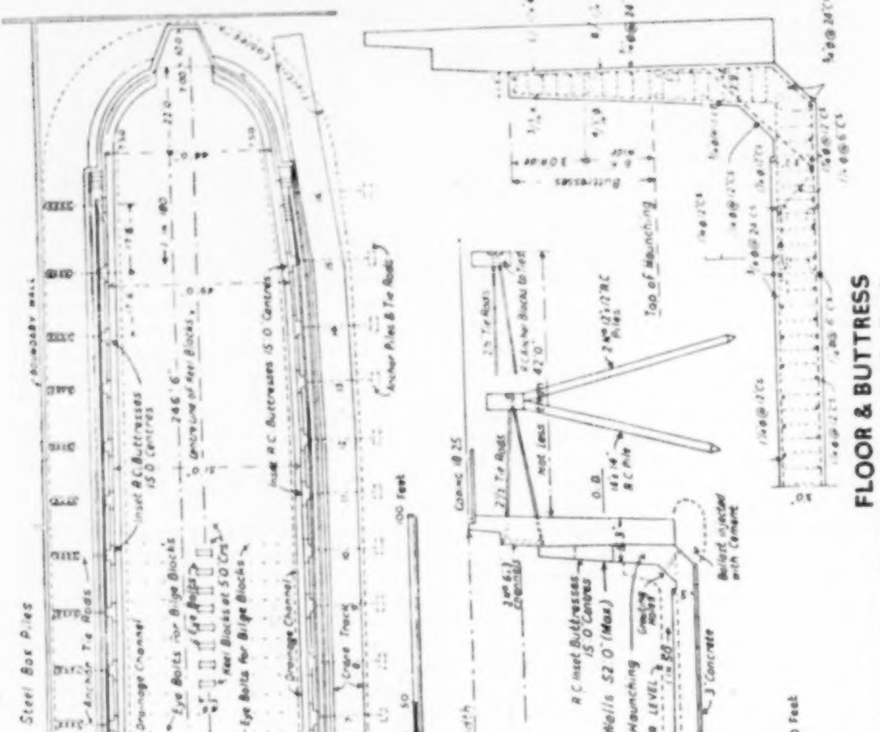

Fig. 1.—Reconstruction of Graving Dock at East India Dock.

and were sufficient to control the water table in the gravel for the whole length of the dock so that there was always a dry surface on which to deposit concrete. Sleeves were provided in the new concrete for the withdrawal of the well-points.

On the east side of the dock the buttresses are connected to groups of three reinforced concrete raking piles by a pair of 2½-in. diameter steel tie-rods with screwed couplings. The piles also carry a concrete beam to take one rail for a travelling crane. The space available on the west side of the dock is not more than 12 ft. wide and is insufficient for the provision of raking piles. The tie-rods from each buttress are attached by a steel bridle to steel box-piles driven vertically into the gravel in groups of five. The bases of the walls and the box-piles beneath the remaining part of the old wall are injected with cement grout to consolidate the gravel, which might have been loosened during the excavations. The reconstruction was executed by day and night work throughout. The contractors were Messrs. George Wimpey & Co., Ltd.

Inflatable Rubber Tubes.

A PROPRIETARY type of inflatable rubber tube, of American origin, for forming holes in concrete, is now being made in this country. The tubes are available in lengths up to 60 ft. and in diameters which, when inflated, form round holes of 1 in., 1½ in., 2 in., and 3 in. diameter. Larger holes may be formed by tying together a number of tubes.

The tubes are placed in the shuttering and supported if necessary, and inflated by an air pump at a pressure of from 40 lb. to 70 lb. per square inch. The concrete is then placed, and when it has hardened sufficiently a valve is opened and the tube is deflated and withdrawn. The valve is of the type commonly used for pneumatic tyres.

A feature of the tube is the ease with which it can be freed from the concrete and withdrawn. When the tube is inflated it contracts in length as it increases in diameter; the reduction in length is about 15 per cent. The rubber tube has a braided fabric core with a helically-wound wire and two layers of helically-wound fibres. One of the layers of fibres is wound at a greater tension than the other. When the tube is inflated it twists and when subsequently deflated and pulled at one end in order to withdraw it, the reduction of diameter causes the fibres and the wire to untwist the tube so that it is freed from the concrete by the decrease in diameter and the twisting motion. A very slight pull only is necessary to release a deflated tube and to allow it to be pulled out, even when

Fig. 1.

the tube embedded in the concrete has a right-angle bend or is looped so that both ends are together at one end of the member and the length of the loop is 6 ft. or so.

In addition to their use in forming hollow cores to lighten precast concrete products such as floor beams, these tubes may also be used for forming holes for services such as are required in lighting poles. They may also be used for forming holes for electricity and other services in in-situ concrete, and for forming holes for cables in prestressed concrete beams. The tubes do not need oiling before use, and it should be possible to use them

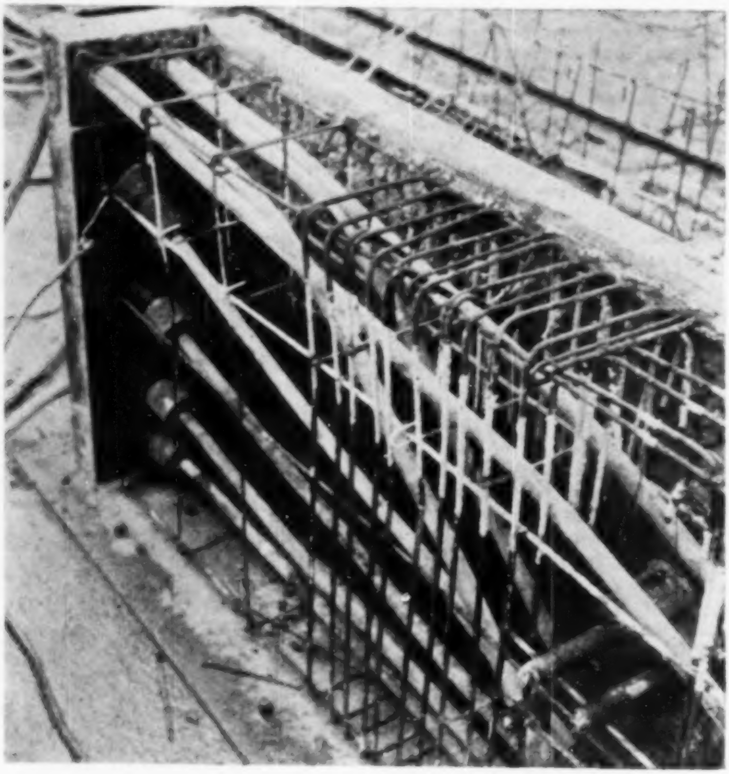

Fig. 2.

hundreds of times before they are worn out.

Use of Tubes in Prestressed Concrete.

In France, M. Freyssinet has recently made some tests with these tubes from which it has been found that they can be successfully used for forming ducts in concrete in which prestressing cables are subsequently inserted. Tubes having an external diameter of ⅞ in. when deflated have been found to be suitable for use for anchorages and cables of the Freyssinet type having twelve to eighteen wires.

In one test a 13-tons beam (*Figs.* 1 and 2) containing 130 ft. of twelve-wire and eighteen-wire cables was cast using these tubes. The inflated tubes were placed in the shuttering in the same way as are sheathed cables. It was found that when tied with wire at about 4-ft. intervals the inflated tubes were sufficiently rigid to maintain the curves and alignment required. The inflated tubes gripped the anchorage cones, through which they passed, and made a watertight joint, and it was unnecessary to apply bitumen or putty jointing. A few hours after casting the beam, the tubes were deflated and consequently lengthened and untwisted, and were easily withdrawn from the concrete by one man. The cables were inserted in the hole thus formed without difficulty.

The trade name of the tube described is "Ductube," which is now made in England and stocked by Messrs. Wiggins-Sankey, Ltd. A rubber core is also used in the Magnel system of prestressed concrete, but this core is not inflated.

Correspondence.

ENGINEERING EDUCATION.

SIR,—May I join issue with you on the subject of "The Universities and Technology" which you discuss editorially in your January issue? I am entirely in sympathy with what I suppose to be your aim—to train engineers in a broader and more liberal way—but I take exception to some of your statements.

You have much sympathy with the view that the University should be a true seat of learning, that is that its teaching should be confined to the arts. Surely this is using the word "learning" in too narrow a sense? We are agreed, I suppose, that "learning" is to be acquired so that the learned one may take his place in society as an educated citizen. To this end he must not only have mastered a body of knowledge, but he must have had some training in using it creatively. In these days the body of knowledge is dangerously incomplete unless it includes some science. Training for constructive thinking should be obtainable by a proper study of any subject normally taught in British Universities to-day; in practice, however, sterile criticism masquerades as thinking in too many arts departments.

In a later passage you say that "an arts degree should be an essential qualification for admission to a course of science or engineering". If this remark, together with the one quoted in the previous paragraph, means that you want students to go to a University and read arts, and then proceed elsewhere to study engineering, then I am not in agreement with you. I will not press the point, however, since I may be misinterpreting you.

As you are aware, the practice is not uncommon in America for the early university years of engineering and science students to be spent in studying the humanities almost exclusively. The account of the system at Columbia, recently published by Professor Finch ("Trends in Engineering Education: The Columbia Experience." Columbia University Press, N.Y.) describes such an arrangement in a way that makes it sound most attractive. I am pretty sure, though, that this idea has developed in America because of the comparative weakness of school education there by academic standards.

That brings me to my final point. The education of engineers must be considered as a whole, taking into account at least the later school years as well as the years spent on technical work. If specialisation at school were avoided, and boys studied a reasonably well-balanced range of subjects before starting their engineering work, then I think that the troubles to which you draw attention would largely vanish.

You have done a service in ventilating this matter in your columns, but I cannot resist saying that a profession which can indulge in such a remarkable display of self-criticism as has recently taken place cannot be so blind to the wider aspects of life as you imagine.

J. A. L. MATHESON.

Professor of Civil Engineering,
The University of Melbourne,
Australia.

SIR,—I have read with interest the Editorial Note in your May number with which, in common with a modest number of other civil engineers, I largely agree. This broad method of training of a young engineer has always been the practice at Dublin University, which houses the oldest engineering school. There a young aspirant must first obtain at least a Pass Degree with Professional Privileges in Arts, before he can obtain the Engineering Degree.

I regret, however, that you should make any derogatory statement at this time in connection with the Engineers' Guild, for the earlier its ascent to power the sooner will the profession improve from all aspects, general culture included.

W. D. KINGSTON.

Public Works Department,
Kano, Northern Nigeria.

[Our reference to the Engineers' Guild was not, and was not intended to be, derogatory to that body, with whose aims we have much sympathy. Our comment dealt only with a statement of one of its members printed in the Journal of the Guild.—ED.]

Storage of Portland Cement in Paper Bags.

EXPERIMENTS made by Rocla, Ltd., of Australia, on the effect of covered, but not air-conditioned, storage of cement on the strength of 3 : 1 cement mortar showed that there was a slow deterioration of cement when stored in paper bags. The rate of loss of strength was about 4 to 5 per cent. per month of storage.

Six 5-ply paper bags of cement were taken at random from a consignment. One bag was opened immediately and the contents tested. Then it was closed, wrapped in polyvinyl cloth, stored in its wrapping, and tested again after six months. The other five bags, which were not wrapped in polyvinyl cloth, were stored in a dry room and one was opened and tested every 28 days. The room was not air conditioned, and so conformed more or less to the fluctuations of humidity and temperature outside.

The total reduction of strength of the cement in the first bag, after storage for six months wrapped in polyvinyl cloth, was 5·5 per cent. after storage of the specimen in water for seven days, 12·5 per cent. after 28 days' water storage, and about 10 per cent. after 28 days' combined storage (1 day in damp air, 6 days in water, and 21 days in air). The strength of specimens made from the cement stored in unwrapped bags was reduced by 31·5 per cent. at seven days, 25·8 per cent. after 28 days' wet storage, and 23 per cent. after 28 days' combined storage, the cement being stored for six months.

In describing these tests (in the Journal of the American Concrete Institute for December, 1949), Rocla, Ltd., report being told that similar experiments, in which cement was stored in bags in a shed such as a contractor might use on a site, showed a loss of compressive strength of 28 per cent. after storage for six months, 40 per cent. after one year, and 54 per cent. after two years. The composition of the concrete specimens in this case was 1 : 5 by weight. It was also found that some of the loss of strength was recovered by storing the concrete specimens for some time. For example, when tested at 28 days, the compressive strength of concrete made with cement stored for six months in paper bags was reduced by 25 per cent., whereas when tested at six months the reduction was only 16 per cent.

A New Piling Code in New York.

THE authorities of New York City have recently adopted a piling code that bases the permissible load on tests instead of on formulæ. As the tests in many cases permit higher loads, it is said that the cost of foundations can be reduced by 25 per cent. Load tests on piles were required by most previous codes, but whereas in the past tests were used as a check on the ability of the pile to carry a calculated load, tests are now made in advance to determine the working load. Under the old rules the permissible working load might be reduced, but never increased, as a result of the tests.

Difficulties of compiling a satisfactory code arise chiefly from the fact that soils to which or through which piles are driven are rarely uniform. In the New York code a few short clauses specify the principles of the test, but about 6000 words are used to restrict the requirements against dangers due to non-uniformity of the ground.

PILES FOR SMALL STRUCTURES.—The new code admits that tests cannot be applied economically to piles for small structures or to large buildings when the time for starting the work is limited. For such conditions, maximum loads are specified and are calculated by the "Engineering-News" formula which is known to be safe in New York because of the experience of more than 40 years. The limitation to New York is essential, because the writers of the code have no faith in this formula if it is applied to any class of soil.

COLUMN-STRENGTH OF PILES.—The code assumes that piles embedded in ground having any bearing value, that is any soil that is not fluid in its undisturbed condition, act as short columns regardless of their actual length, and arbitrary values for timber, concrete, and steel piles are specified. These values cover the first requirement for a pile, that is the ability of the shaft to withstand safely the working load. The next consideration is the greatest load that can be permitted from the point of view of bearing.

TESTS.—One bore or pit, extending sufficiently far into ground of good bearing value to establish its nature and thickness, is required for every 2500 sq. ft. of building area. For structures of more than one story (except dwellings of two stories and buildings where the load exceeds 1000 lb. per square foot) at least one boring in each 10,000 sq. ft. must extend 100 ft. below the level of the surface, or 25 ft. into one type of soil as good as loose sand, or 5 ft. into rock. Areas of similar soil conditions are delineated over the entire site and in each area three piles of the proposed type must be driven. For piles not greater than 8 in. in width, a hammer delivering a blow of at least 7000 ft.-lb. is required, except that the least blow for a pile to carry 25 tons or more is 15,000 ft.-lb. For piles more than 8 in. but not greater than 18 in., a blow of 15,000 ft.-lb. is required and for larger piles 22,000 ft.-lb. The driving record of the three piles is taken to indicate the correctness of the assumption of the similarity of the area.

One of the three piles is tested by loading, but a test must be made on at least one pile for each 15,000 sq. ft. of building area and on at least two piles in each structure. The test load must be twice the proposed working load and is applied in seven increments of $\frac{1}{2}$, $\frac{3}{4}$, 1, $1\frac{1}{4}$, $1\frac{1}{2}$, $1\frac{3}{4}$, and 2 times the proposed working load. The settlement and recovery must be recorded to 0·001 ft. for each increase of load. The load may be increased after there has been no settlement for two hours. The greatest load must remain in place until the settlement does not exceed 0·001 ft. in 48 hours. The load may be removed in decrements not exceeding one-quarter of the total test load at intervals of one hour.

WORKING LOAD.—The greatest permissible working load is that which causes a net settlement of not more than 0·01 in. per ton of total test load, or one-half of the load which causes a gross settlement of 1 in., whichever is less. Net settlement is defined as the total settlement under load minus the recovery of the pile after the load is removed. Working loads are also limited by provisions for different types of piles.

FRICTION PILES.—Within the limits given later, the greatest permissible load on a friction pile is 30 tons if calculated by a driving formula, and 60 tons if proved by tests. For friction piles the total load is assumed to be distributed

by the lower two-thirds of the length of the pile. Therefore the cross-sectional area required to carry the total load must be provided from the upper third point to the head of the pile.

END-BEARING PILES.—End-bearing piles driven to rock, or hardpan or boulder-gravel directly overlying rock, may be used for loads up to 40 tons if they are driven to the requirements of the formula. If they are tested to twice the design load they may be loaded to the permissible stress, but the load must not exceed 120 tons for a pile bearing on rock, or 80 tons for a pile bearing on hardpan or boulder-gravel directly overlying rock.

For end-bearing piles 75 per cent. of the load can be assumed to be carried at the toe of the pile if the pile exceeds 40 ft. in length. For shorter piles the total load is considered to be carried at the toe. The part of a pile that is free-standing in air or water is considered to be a column fixed at a point 5 ft. below the level of the ground, and 10 ft. below the level of the ground if the soil is poor.

CONCRETE PILES.—Rules are given for timber, steel, precast concrete, and cast-in-situ concrete piles. The permissible stress in concrete is one-quarter of the compressive strength at 28 days, but must not exceed 1000 lb. per square inch.

Precast concrete piles must have longitudinal reinforcement equal to at least 2 per cent. of the volume of the concrete. Lateral reinforcement must be at least ¼ in. diameter and spaced at not more than 12 in. centres throughout the length of the pile except for a length of 3 ft. at the bottom and top where the spacing must not exceed 3 in. The cover of concrete must be at least 2 in. For reinforcement not exceeding 4 per cent., the modular ratio is assumed to be 30,000 divided by the compressive strength of the concrete.

JACKED PILES.—Piles may be driven by jacking without impact and the working load on such piles must not be more than half the force required to drive them, the actual carrying capacity being proved by test. For piles in underpinning, the temporary working load must not exceed the force required to drive the pile. The working load on permanent underpinning piles may be up to two-thirds of the jacking force required to cause the penetration if this force is maintained for ten hours; otherwise the permissible load is half the total load for penetration.

PILES IN GROUPS.—Piles in groups must be spaced at not less than twice the diameter, or 1·75 times the diagonal, of the pile. The spacing must not be less than 24 in. for piles bearing on rock or 30 in. for other piles. Piles in groups, or adjacent groups of piles, founded on most types of sands, gravels, clays, and inferior soils, must be spaced 10 per cent. farther apart than the foregoing for each interior pile up to a maximum of 40 per cent.

If the ground, driving, or load tests indicate that there is an uncertainty that a group of piles will carry the permitted load on a single pile multiplied by the number of piles in the group, a load test up to 1½ times the working load on the group may be required.

MISCELLANEOUS NOTES.—Lateral loads in excess of 1000 lb. per pile are not permitted unless it is proved by test to twice the proposed load that the lateral movement does not exceed ½ in. and that under the working load the movement is not greater than $\frac{3}{16}$ in.

A tolerance of 3 in. from the true position of a pile is allowed. If piles are out of plumb more than 2 per cent., the design must be modified.

A jetted pile must be driven for the last 3 ft.

Splices that cannot be inspected after driving must develop at least 50 per cent. of the strength of the pile in bending.

Copies of the new code and of an article by Mr. J. H. Thornley in "Engineering News-Record," from which the foregoing notes have been abstracted, may be obtained gratis from Mr. Thornley, 2 Park Avenue, New York, U.S.A.

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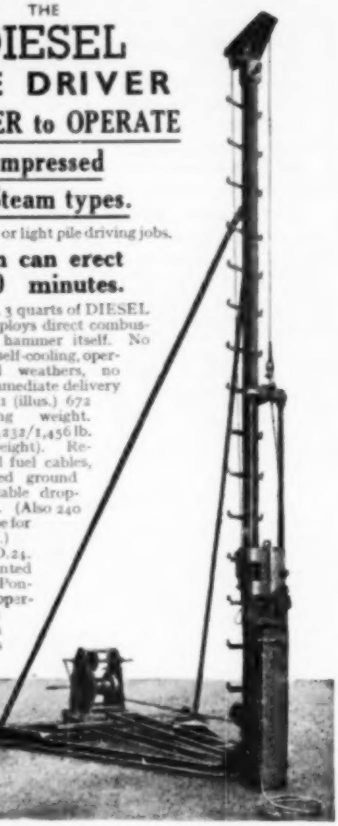

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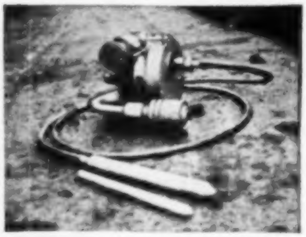

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